

A Summary of the Illinois Skid-Accident Reduction Program 1964-1980

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A SUMMARY OF THE ILLINOIS SKID-ACCIDENT
REDUCTION PROGRAM - 1964-1980

Prepared for the Illinois Department of Transportation
Bureau of Materials and Physical Research

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A SUMMARY OF THE ILLINOIS SKID-ACCIDENT REDUCTION PROGRAM - 1964-1980

CHAPTER ONE

INTRODUCTION

This report summarizes the activities of the Illinois Department of Transportation during the years 1964-1980 in conducting an extensive and systematic program of skid-accident reduction initiated in 1964. Most of the work has been accomplished in cooperation with the Federal Highway Administration and in compliance with Federal requirements and directives. Most of the information contained in the report regarding the Illinois activities is drawn from more detailed reports issued by the Department from time to time on various phases of the work effort. A lesser amount is drawn from unpublished reports and memoranda of the Department. Background information of a more general nature contained mostly in Chapter 1 has come from such sources as the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the Transportation Research Board. Principal source reports are listed at the conclusion of this summary report.

Adequate resistance of pavement surfaces to skidding is an important and essential factor in the maintenance of vehicle control. Unfortunately, not all pavements provide the necessary degree of skid resistance, especially during wet weather when pavement surface friction is at its lowest level, and a disproportionate number of accidents involving skidding occur.

Recognizing that a good potential existed for improving highway safety in Illinois through upgrading the frictional characteristics of its pavements,

the Illinois Department of Transportation in 1964 undertook a systematic approach toward improvement in this problem area. Recognizing also the very limited availability at the time of practical knowledge regarding the design and construction of skid-resistant surfaces, the initial effort was concentrated in the field of research and development. As the body of knowledge grew, Statewide sampling studies were undertaken to determine more exactly the nature and extent of the problem in Illinois, and experimental studies were started to establish better means for assuring adequate skid resistance in pavement surfaces. With the acquisition of further knowledge, pavement design, construction, and maintenance practices were reviewed and changes in practice introduced where conditions warranted. Additionally, a program for detecting and correcting locations with a high incidence of wet-weather accidents was introduced. All elements in the skid-accident reduction program are of a continuing nature.

The purpose of this report is to establish a single document containing sufficient information gleaned from existing published documents, internal reports, and readily available file material pertinent to skid-accident reduction activities in Illinois to permit a thorough overall examination and evaluation by individuals within the State and FHWA with the intent of recognizing further improvements for practical application and of developing a sound total future skid-accident reduction program for Illinois.

HISTORICAL

Slippery pavements have been known for most of the years of motor vehicle use to have the potential for causing accidents. Over half a century ago certain pavement surfaces were being recommended over others as having relatively better frictional characteristics. Even the oldest

of today's drivers is likely to remember from the beginning of his driving experience the "Slippery When Wet" signs attesting to failures in providing adequate skid resistance.

As vehicles became more numerous, and highway and vehicle design improvements produced higher driving speeds and more rapid accelerations and decelerations with attendant increases in tire-pavement frictional demand, the number and severity of skidding accidents increased to the extent that they became by the 1950's a major cause of concern. However, it was not until the latter part of that decade and the early 1960's that the development of testing equipment essential to the measurement and comparison of the frictional properties of pavement surfaces, and to learning the frictional requirements of traffic, had reached a stage where truly effective evaluations of these properties could begin. Prior to the development of this equipment, the only signal of insufficient skid resistance was an excessive number of skidding accidents.

MEASUREMENT OF PAVEMENT SURFACE FRICTION

In connection with the tire-pavement system, friction is defined as the force developed when a tire that is prevented from rotating slides along the pavement surface. Although usually thought of as a pavement property (and used in that context in this report), skid resistance, as a friction process and in common with all friction processes, actually is a function of the material properties of both pavement surface and tire, and of prevailing operating conditions.

Not only are the factors that impact on friction numerous, but the skid resistance of the pavement surface itself is a highly variable characteristic. Nevertheless, the friction levels of pavement surfaces

must be measured, and measured under controlled conditions, if friction quality is to be brought within reasonable bounds. In addition, the friction levels required by traffic under whatever needs prevail also must be determined.

The main principles of pavement skid resistance generally were known at the time Illinois entered its systematic program of skid-accident reduction. Also, a number of friction testers in various stages of development were in existence. The basic method for determining skid resistance then, as now, consists of measuring the force required to drag a tire over a prewetted pavement. Most such testers then and today are trailers with standardized tires pulled over the wetted pavements in the locked-wheel mode. In the United States, the test tires are defined by ASTM Standard E-249 and test procedures are described by ASTM Method E-274. Few of the test trailers, however, conform to identical plans.

Major advancements have been made in the past several years in Illinois and elsewhere in the development of friction-testing devices. The frictional characteristics of pavement surfaces in the locked-wheel, straight forward mode now can be measured with considerable precision by several methods. Friction characteristics so measured can be related empirically in a general way to the needs of traffic and drivers.

The initial actions taken by Illinois as it entered upon a specific program of pavement friction improvement were the development and the construction of a reliable device for measuring the frictional characteristics of pavement surfaces(1),(2).^{1/} This work was done as the first phase of a

^{1/} Numbers in parentheses refer to references listed at end of report.

broad research project, IHR-86, Skid Resistance of Pavement Surfaces, conducted in cooperation with the Federal Highway Administration with the following stated objectives:

- (1) To develop new equipment or to improve existing equipment for determining the friction characteristics of highway pavements, intersections, and interchanges.
- (2) To determine the friction characteristics of existing highway pavements.
- (3) To study the polishing characteristics of aggregates used in pavement surfaces.
- (4) To develop durable and economical means of increasing pavement friction.
- (5) To assemble a more positive body of knowledge concerning pavement friction characteristics for incorporation in highway design and safety policies.

Friction testers in use at the time Illinois began its research project were of three general types: (1) portable hand-operated; (2) full-size stopping-distance vehicles; and (3) test trailers pulled by vehicles. The test trailers appeared to best fit the heavy-duty needs foreseen by Illinois and the choice was made to further develop and construct this type of testing device. The test criteria established by ASTM Committee E-17 and described in ASTM Method E-274 were followed in the development and construction of the device. The total system includes both a two-wheel test trailer and a permanently paired towing vehicle. Tires as specified by ASTM Standard E-249 are mounted on the trailer. For measurement, the trailer is towed, usually at a speed of 40 mph, over pavement on which

water is applied in front of the test tire. The test wheel is locked by braking for a certain distance and the torque of the test wheel then measured and recorded for a specified length of time. The result of the test is reported as a friction number (FN).^{2/}

Although the friction number determined at 40 mph (FN₄₀) is that which is most commonly used, friction numbers determined in series at different speeds also have been considered by some investigators to be useful in assessing the adequacy of the macrotexture (a particularly important contributor to skid resistance in high-speed travel in wet weather). Using friction measurements at different speeds, friction number-speed gradients can be developed to show the relationship between friction number and speed. For high-speed travel, a gradient with a low rate of descent is a desirable characteristic in addition to a high FN₄₀. Friction number-speed gradients are discussed more fully in following chapters of the report.

Friction number-traffic use curves developed from successive measurements can show the rate of wear or loss of skid resistance with traffic use. Here, a curve of little downward slope suggesting minimum wear is desirable.

The test-trailer that evolved from the early Illinois work was completed and became fully operational in 1969. It has been in continuous use since that time in surveying the frictional characteristics of thousands of miles of pavement. As needs increased, a second and similar test system was added in 1977.

^{2/} Formerly termed skid number (SN).

The locked-wheel trailer test method of ASTM E-274 in use by Illinois and many other states is now considered to be the most convenient, and in the long run the most economical, method for survey. Combined with operation of the FHWA-sponsored Field Test and Evaluation Centers for friction testers, the method provides for the acquisition of reliable measurement data and correlation nationwide.

FRictionAL REQuIREMENTS

The level of skid resistance of a pavement surface never can be too high from a safety viewpoint. However, any attempt to provide exceedingly high frictional levels in pavements without a consideration of cost cannot be justified economically. To be meaningful, friction standards must be coordinated with the needs of traffic. Because of the almost unlimited number of variables involved and the complexities of the relationships that exist between them, no precise minimum skid-resistance standards for pavement surfaces ever have been determined. This was true in 1967 when NCHRP Report 37, "Tentative Skid-Resistance Requirements for Main Rural Highways,"(3) was issued and is still true today.

Nevertheless, for further advances to be made in conquering the skidding problem, it has been expedient to introduce simplifications that allow the establishment of tentative minimum friction-quality standards. Customarily, this is done by defining traffic requirements in terms of frictional quality as measured by the standardized locked-wheel trailer equipped with the specified treaded tire dragged over a wetted pavement at a prescribed test speed of 40 mph (FN_{40}). This necessary simplification disregards the known significant variation of the frictional needs of traffic with speed, the variation of the measured values of friction with speed, and other variations as well.

NCHRP Report 37, as a result of many considerations, proposed that $FN_{40}=37$ be accepted tentatively as the minimum permissible standard for main rural highways. The value of $FN_{40}=37$ assumes a mean traffic speed of 50 mph. The full range of tentative interim friction-quality requirements is included in the following table from the report.

TENTATIVE INTERIM SKID-RESISTANCE REQUIREMENTS
FOR MAIN RURAL HIGHWAYS*

Traffic Speed (mph)	Recommended Minimum FN^{**}	
	Measured at Traffic Speed	Measured at 40 mph
30	36	31
40	33	33
50	32	37
60	31	41
70	31	46

* From Table 18, NCHRP Report 37. These values are recommended for main rural two-lane highways. For limited access highways lower values may be sufficient, whereas certain sites may require higher values.

** FN = friction number, measured according to ASTM Method E-274.

Although the values of the preceding table were derived rationally on the basis of available data and information, they were offered as recommendations and remain recommendations 13 years later. Various jurisdictions around the country frequently have adopted their own guidelines that, on the whole, do not differ much from the recommendations of NCHRP Report 37. No agency, including the Federal government, has to date considered the advancement of knowledge in the area to be sufficient for the establishment of firm and binding standards.

Measured friction numbers in the 10 to 20 FN range generally are recognized to indicate an undesirably high potential for skidding accidents.

Friction numbers above 40 are almost invariably recognized to be sufficient. Uncertainties have been centered primarily within the intermediate range between 20 FN and 40 FN, the very range in which the friction numbers of a majority of Illinois pavements have been found to lie.

The process of selecting tentative minimum friction numbers among jurisdictions seems largely dependent on data currently available, assumptions made in analytical treatments, a degree of intuition based on experience, and always the judgment of the individuals attempting to establish minimum numbers.

Established minimum levels for friction quality, even though tentative, are needed for designing and constructing pavements of adequate skid resistance, and supplement accident records in selecting pavements for restoration of skid resistance. The tentative minimum of $FN_{40}=37$ as recommended in NCHRP Report 37 is accepted by Illinois engineers as a reasonable value for satisfying frictional demand when the mean vehicle speed is 50 mph as demonstrated in the report. However, the application of a single minimum FN_{40} value under all conditions has not been viewed as appropriate for Illinois. In common with most agencies dealing with the skid problem, Illinois has drawn considerably from its own experience by the processes outlined in the preceding paragraph and has established its own tentative friction guidelines for evaluating pavements, based on testing at 40 mph in accordance with ASTM E-274. Three levels of friction quality have been established tentatively for assessing the frictional quality of pavement surfaces as follows:

<u>FN_{40}</u>	<u>Adjective Description</u>
Over 36	Satisfactory
30 through 36	Marginal
Below 30	Low

It is believed that, when the FN_{40} value is 37 or over, some condition other than lack of sufficient friction is the primary cause of accidents. When the FN_{40} value lies in the range of 30-36, uncertainty is believed to exist as to whether pavement friction is a primary factor. At FN_{40} values below 30, pavement friction is considered to be a probable contributing factor.

Experience has shown that pavement friction usually deteriorates as traffic wears and polishes the surface. The rate of wear and polishing varies with the amount and composition of traffic and the kind of materials in the pavement. It is not at all infrequent for the frictional life of a pavement surface to be well below the structural life of the pavement. In recognition of the deterioration problem, it usually is not considered wise to construct a pavement surface at or only slightly above the minimum desirable friction level. Unfortunately, the development of practical information on deterioration rates is a difficult and time-consuming process that has not yet produced sufficient information for establishing limiting values of surface friction at construction to assure attainment of a selected service life.

SKID ACCIDENTS

Accident statistics thus far have not proven useful in establishing minimum friction values for pavement surfaces(4). The few studies that have been made have permitted conclusions of doubtful validity at best. Although the highway accident problem is indeed serious, accidents actually are rare events from a purely statistical standpoint. As rare events resulting from a multitude of possible causes, the degree of sophistication needed in the control of sample selection and the accuracy and uniformity of

accident reporting for sound statistical analyses leading to the establishment of minimum friction values has not been attainable to date.

Whether or not this can be achieved in the future remains to be seen.

Accident statistics for specific sites, on the other hand, have been very useful when applied in conjunction with friction measurements to determine whether low skid resistance has been a major contributing cause of accidents at the sites in question. Where skidding-accident rates are unduly high, accident sites often can be identified by a simple inspection of pin maps and accident reports for the sites. High accuracy in the data is not necessary. Where the pavement friction contribution to accidents is less well-defined, as is often the case, greater accuracy in the data and a more precise treatment of the data are required. This is particularly true in the establishment of priorities for improving pavement friction where needs seem invariably to exceed available funds. The procedure used by Illinois in selecting skid-prone sites for treatment is described in Chapter 4.

Accident data also are essential and currently usable in assessing and validating the effectiveness of skid-resistance treatments that are applied.

PROVIDING SKID-RESISTANT PAVEMENT SURFACES

The resistance of pavement surfaces to skidding is dependent on many factors including characteristics of the component materials, mixture design, construction methods, climatic exposure, and the nature and volume of the traffic to which they are subjected. Among the many factors that affect skid resistance, the microscopic and macroscopic roughness (also

referred to as "microtexture" and "macrotexture") of the surface and of the mineral aggregate particles it contains, appear to have the greatest influence. Microtexture is what makes aggregate particles seem rough or smooth to the touch. Macrotexture provides the channels by which water can escape from under the tire to allow tire contact with the pavement in wet weather(4). The terms "microtexture" and "macrotexture" are used without a clear dividing line between the two. One source places the division at about 0.01 in. and places an upper limit on macrotexture at about 0.5 in.(5).

Experience has shown that almost any dry pavement reasonably free of dust and loose material will provide adequate skid resistance. Insufficient skid resistance is primarily a wet-weather problem (consideration of frost, ice, and snow-covered surfaces excluded). The friction quality of dry pavements is nearly independent of speed. This is not true for wet pavement surfaces where the quality of friction can reduce significantly as vehicle speed increases. Of the two functions of texture in wet weather (resistance to sliding as the tire passes over the textural irregularities and facilitation of the flow of water from the tire-pavement interface), that of providing escape channels for the water is more important.

The macrotexture that produces the escape channels for water in wet weather may serve the additional function of controlling the hydroplaning phenomenon that occurs when a tire moves so fast over a fairly thick layer of water on the pavement that it "rides up" and loses all contact with the pavement surface(4).

Texture in a bituminous surface is most significantly influenced by aggregate size, gradation, and other aggregate characteristics. Texture

in a portland cement concrete surface is most significantly influenced by the texturing method used in the finishing process.

Although much is known about the influence of the surface texture of pavements on skid resistance, the characteristics involved have been too numerous for the development of ways to describe it quantitatively.

In a dense-graded bituminous concrete pavement as used in Illinois, macrotexture is provided by the coarse aggregate and microtexture by both the fine and the coarse aggregates. Good macrotexture is considerably dependent on mix design which should provide for a maximum exposure of coarse aggregate particles. The durability of the surface texture is a function of the abrasion and polishing characteristics of the coarse aggregate, the ability of the mix to resist consolidation, and the volume of traffic(5).

In the open-graded asphalt friction course (OGAFC) that was introduced a few years ago because of perceived advantages in controlling both wet-weather skidding and hydroplaning, and which has been the subject of considerable experimentation in Illinois, the attainment of the desired macrotexture is primarily a function of aggregate gradation and asphalt content. The microtexture must be provided by the coarse-aggregate fraction without assistance from fine aggregate, with the result that this characteristic must receive careful consideration in selection of the coarse aggregate. Durability characteristics of the coarse aggregate are also a matter for careful consideration.

In a portland cement concrete pavement, fine texture is contributed by the fine aggregate and coarse texture by the finishing process. Durability of the surface texture is a function of the wear-resistant qualities

of the concrete and the character and volume of traffic(5). Quality concrete is obtainable under current specifications. It is worth noting, however, that the wear resistance of concrete increases as the cement factor is increased and as the water-cement ratio is decreased. Also, a fine-aggregate content near the upper practicable limit is desirable(5).

The improvement of the frictional quality of pavement surfaces is only one of several means that need to be considered for reducing wet-weather pavement skidding accidents in a skid-accident reduction program. Although improvement of the pavement surface to meet the foreseeable needs of traffic provides the most direct approach, lessening traffic needs sometimes also can provide a viable approach. The application of speed restrictions and the use of warning signs, better driver education and enforcement of speed limits, elimination of items that cause traffic turbulence, removal of hazards, and provision for quick removal of water from pavement surfaces during rainstorms all can contribute to the reduction of the frictional needs of traffic.

FEDERAL SKID-ACCIDENT REDUCTION PROGRAM

As one part of a Federally mandated Highway Safety Program, each State is expected to develop and manage a skid-accident reduction program. According to FHPM 6-2-4-7, dated December 10, 1975, the purpose of the skid-accident reduction program is to reduce skidding accidents on wet roads through the application of a systematic plan for the identification and correction of sections of roads with high or potentially high skid-accident locations. This systematic plan, according to FHPM 6-2-4-7, should include at least three basic activities:

- (1) The evaluation of pavement design, construction, and maintenance to ensure that only pavements with good skid-resistance characteristics are used in construction and resurfacing.
- (2) The detection of locations with a high incidence of wet-pavement accidents by utilizing the State accident record system and local accident record system as applicable, and the development of priorities for correction of the locations.
- (3) The analysis of skid resistance for all roads with a speed limit of 40 mph or greater, so that skid resistance can be given consideration in the development of priorities for resurfacing and maintenance programs.

FHPM 6-2-4-7 provides detailed guidelines for pavement friction measurements to be made by States as part of the skid-accident reduction program and, in addition, furnishes information on a broad range of activities that are expected to be a part of each State's skid-accident reduction program.

The most recent and most comprehensive overview of the factors that should be considered as elements of any skid-accident reduction program in the view of the Federal Highway Administration is offered in FHWA Technical Advisory T 5040.17, dated December 23, 1980. The stated purpose of this Technical Advisory is to provide guidance for State and local highway agencies in conducting skid-accident reduction programs.

Because of its importance in evaluating the activities and content of the Illinois accident-reduction program as summarized in this report, and in planning for the future, Technical Advisory T 5040.17 is included in its entirety in this report as Appendix A.

ILLINOIS SKID-ACCIDENT REDUCTION PROGRAM

As indicated earlier, the entry of Illinois into a systematic program of skid-accident reduction began in 1964 with the establishment of a comprehensive and multiphased research project in the area of skid resistance. Completion of the development and fabrication of a test trailer for measuring the frictional characteristics of pavement surfaces in the field in 1969 provided an essential tool for surveying and assessing the quality of the frictional characteristics of existing Illinois pavements and for evaluating the results of field experimentation to develop more skid-resistant surfaces. With the establishment of the Federal skid-accident reduction program in which all States participate, the Illinois program was broadened to include all of the basic activities prescribed in FHPM 6-2-4-7 as identified previously.

The initial survey of existing conditions of Illinois pavements made in 1969-1971 consisted of a general inventorying on a sample basis of all the major types of road surfacings in use. Field work included 8,300 individual friction tests at over 400 sites throughout the State. Summary details of the initial inventory sampling survey are given in Chapter 2. More complete details are given in reports (6), (7), and (8).

The field experimentation has included numerous trial installations on construction projects intended to provide information useful in improving skid resistance through better pavement design, construction, and maintenance practice. Begun in 1965, these studies have been concentrated for the most part in three principal areas: (1) the frictional characteristics offered by different types of coarse aggregate components in dense-graded

bituminous concrete mixtures and by the sands in sand-asphalt mixtures; (2) the general behavior and frictional characteristics of open-graded asphalt friction courses with coarse-aggregate type being a major variable; and (3) the influence of texturing practice on the frictional characteristics of portland cement concrete pavement.

Analytical studies of data assembled in the inventory sampling study led to the association of certain coarse-aggregate types in use in bituminous concrete pavements with less-than-desirable frictional characteristics. On the basis of this information, and with information acquired in the experimental studies, several changes were made in pavement design and construction policies to upgrade the requirements for coarse aggregates to be used in bituminous concrete mixtures. A similar application of information has led to the adoption of a more positive means for texturing portland cement concrete pavements to assure better frictional characteristics.

Inventory samplings have continued since the initial survey was completed in 1971, but on a less intensive basis pending the careful development of a comprehensive sampling plan that can be followed without need for appreciable revision for many years to come. The sampling study currently is identified as Illinois Project IHR-407, "Pavement Friction Inventory." A second trailer purchased in FY 77 using Federal Hazard Elimination Safety Program funding is now available for friction testing, including the sampling survey testing.

The field experimentation program is of a continuing nature, and promising new materials and processes for improving pavement skid resistance are tested as they become available. Because of the relatively long

life spans normally expected of most pavements, and the economic advantage of introducing frictional qualities that remain acceptable for equal periods of time, many trial installations placed in the experimental program to date are continuing to be observed and evaluated.

Changes that have been made in design and construction policies as a result of findings produced by the studies to date are outlined in Chapter 3. Summary details regarding the experimental work that has been mentioned, and also regarding other experimental activities of a generally less comprehensive nature, also are given in Chapter 3. Evaluations of the effectiveness of changes that have been made to improve friction quality also are discussed. More complete details regarding the experimentation will be found in reports (7), (8), (9), (10), and (11).

Chapters 2 and 3, and also Chapter 4, include some discussions of future activities in the skid-accident reduction program in Illinois where continuation of current activities is likely to be involved as envisioned within the Department of Transportation at the time material was being solicited for the preparation of this report. The discussions are intended to be informative only.

A positive signal of inadequate skid resistance of a pavement is an excessive number of skid-related accidents. Although ideally, friction measurement information should be of a volume and quality to permit correction of deficiencies before abnormal numbers of skidding accidents occur, the time is not yet at hand where this is always possible. Therefore, as in the past, accident recording and analyses are essential components of a skid-resistance management program. Obviously, locations identified as high skid-accident sites require immediate remedial action.

Illinois, since 1973, has used a computer accident information system for both matching and coding accident reports. The overall accident review system includes a specific process for establishing, implementing, and evaluating a program of corrective action. Summary details of the program are presented in Chapter 4. Additional details are presented in a detailed report of the status of the entire highway safety construction program in Illinois issued annually(12).

In the 17 years that the Illinois formalized program of skid-accident reduction has been under way, recorded expenditures for research, experimentation, and friction measurement (nonconstruction expenditures) have totaled \$1,179,000. Construction expenditures for improvement of skid-prone locations have totaled another \$4,400,000. The added costs of a more selective use of aggregates to improve the friction quality of bituminous concrete surfaces, and of other changes in construction and materials use to improve friction quality, although substantial, are difficult to assess. A benefit/cost analysis of friction quality improvement through before-and-after accident studies shows a generally favorable return on the investment.

The skid-accident reduction program in Illinois has been conducted almost entirely in cooperation with the Federal Highway Administration, with Federal participation in the funding at specified matching ratios.

CHAPTER TWO

GENERAL SAMPLING AND ANALYSIS OF THE FRICTIONAL CHARACTERISTICS OF ILLINOIS PAVEMENTS

Successful completion of a reliable frictional-test trailer in 1969 provided Illinois with the measuring element essential to development of an understanding of its pavement-surface friction needs and of how to fulfill these needs. Use of the trailer in a general program of sampling the frictional characteristics of pavements throughout Illinois during the period 1969-71 provided both the understanding needed to approach skid-resistance improvements in Illinois in a logical manner and the knowledge needed to proceed immediately with the implementation of a number of changes in practice to assure better friction quality in new pavements.

Acquisition and study of the friction assessment data, as well as development and construction of the test trailer, and also some field experimentation to be discussed in Chapter 3, all were accomplished under research project IHR-86, "Skid Resistance of Pavement Surfaces" which was active during the years 1964-77. Project IHR-86 was conducted in cooperation with the Federal Highway Administration. Details of the sampling inventory are summarized in this chapter. More complete discussion will be found in reports (6), (7) and (8).

SAMPLING SYSTEM

To compress the field work of inventorying the frictional quality of Illinois pavements Statewide into manageable proportions, a sampling system was devised for the 1969-71 study that was believed could provide a reasonable representation of the system as a whole. Based on previous

knowledge developed mostly by others (and later summarized in (1), five principal factors were considered in sampling selection: (1) pavement type; (2) pavement age; (3) general location; (4) coarse-aggregate type; and (5) traffic volume. These factors were used in the sample selection process as shown in Table 1. Also shown in the table are the numbers of samples in each cell that resulted from a request to the nine highway districts for recommendations. In all, 404 sites were recommended for testing.

It will be noted in Table 1 that the District submittals listed sites for testing in most, but not all, of the categories that were to be considered. Listings from all districts contained portland cement concrete (PCC) sites and dense-graded bituminous concrete (Class I) sites, but not all districts were able to recommend Class B bituminous concrete and Class A surface treatment sites. Overall, a good representation is believed to have been obtained. The number of sites for each pavement type proved to be nearly proportional to the mileage on the Interstate and primary systems, and somewhat less proportional to that on the county system.

The Interstate-primary system in Illinois, as reported in 1975, included somewhat over 13,000 miles of highway, of which approximately 10 percent was Interstate, 29 percent was U S marked, and 61 percent was State marked. Pavement surfaces throughout the system were either portland cement concrete or Class I bituminous concrete with few exceptions. Interstate pavements were primarily portland cement concrete, there being only 100 miles of bituminous concrete. On the primary system, one third of the pavement surfaces were portland cement concrete and two thirds were bituminous concrete.

TABLE 1
DISTRIBUTION OF SITES SELECTED FOR FRICTION-QUALITY SURVEY
(Adapted from (6), Figure 1)

TYPE OF PAVEMENT SURFACE	PORTLAND CEMENT CONCRETE 115 1/				BITUMINOUS CONCRETE				BITUMINOUS SURFACE TREATMENT (Class A) 75			
	CLASS I 174				CLASS B 40							
GENERAL LOCATION	OPEN HIGHWAY 85		STOP INTERSECTION 30		OPEN HIGHWAY 118		STOP INTERSECTION 56		OPEN HIGHWAY 26		STOP INTERSECTION 14	
	GRAVEL 30	CRUSHED STONE 55	GRAVEL 10	CRUSHED STONE 20	GRAVEL 118	CRUSHED STONE 118	CRUSHED STONE 56	CRUSHED STONE 56	GRAVEL 11	CRUSHED STONE 15	GRAVEL 6	CRUSHED STONE 8
KIND OF COARSE AGGREGATE	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅
	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅
TRAFFIC VOLUME	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅
	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₁ V ₂ V ₃	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅	V ₄ V ₅
AGE OF SURFACE (years)	0	1	2	3	4	5	6	7	8	9	10	11
	0	1	2	3	4	5	6	7	8	9	10	11
5-6	2	2	5	6	3	6	1	2	3	1	2	3
	2	2	5	6	3	6	1	2	3	1	2	3
10-12	2	3	1	4	3	3	1	1	3	2	1	1
	2	3	1	4	3	3	1	1	3	2	1	1

TRAFFIC VOLUME LEVELS (1967 ADT)

V₁----- Under 2500
V₂----- 2500 - 4999
V₃----- 5000 and Over
V₄----- Under 400
V₅----- 400 and Over

1/ Numbers are samples in category shown

The county system of highways in Illinois includes over 16,000 miles of mostly low-volume farm-to-market roads with surfacings ranging from PCC to oiled earth. Only the intermediate-type surfacings of Class B bituminous concrete and Class A bituminous surface treatments, of which there are a considerable number of miles, were included in the friction quality study, as shown in Table 1. Of 115 sites tested, 40 (26 open-highway and 14 intersections) were Class B bituminous concrete and 75 (45 open-highway and 30 intersections) were Class A bituminous surface treatments.

Friction tests were made at each site at 40 mph in accordance with ASTM Method E-274. Inner wheelpaths were chosen for testing because experience had indicated that they usually had the most polished surfaces. Open-highway test sites were straight, approximately level one-mile sections. Intersection sites were 500-ft approaches to signals or stop signs. The friction number (FN) recorded for open-highway sites is the average of ten tests in each lane; that for intersections is the average of six tests per lane. When testing four-lane highways, both traffic and passing lanes were tested and recorded separately.

As stated in Chapter 1, Illinois has established tentatively three friction-number levels for analyzing the friction quality of pavement surfaces as follows:

Over 36	Satisfactory
30 to 36	Marginal
Below 30	Low

CLASS I BITUMINOUS CONCRETE PAVEMENT

The Class I bituminous concretes used as surface courses in Illinois are dense-graded asphaltic mixtures with the coarse aggregate components

having nominal top sizes of either 3/8 in. or 1/2 in. The mixtures are designed for a low void content (2 percent). Compaction to a density of not less than 93 percent of the maximum possible voidless density is required. Previous experience with the frictional quality of similar mixtures suggested that the physical characteristics of the coarse aggregate in this mixture would have a major effect on the quality of its friction (3),(4). With this in mind, analyses of the accumulated friction measurement data for the Class I surfaces were centered on this variable and, as anticipated, the coarse aggregates were found to have a strong influence on skid-resistance behavior.

At the time of the study, Class I mixtures in place contained mostly crushed stone coarse aggregates, with crushed gravel being used in the remainder (other than experimentally). Stone from commercial sources in and within practical shipping distance of Illinois is all of sedimentary origin and was classified broadly for the purpose of the study as either dolomite or limestone. Sources of crushed gravel are principally of glacial origin and may contain some igneous in addition to sedimentary material.

To explore the existence of a relationship between coarse-aggregate source and the frictional characteristics of Class I bituminous concrete surfaces, the relative frequency curves of Figure 1 were developed. The crushed gravel and dolomite aggregates will be seen in the figure to have provided a substantially better quality of friction than the limestone aggregates. Considering the three levels of frictional quality mentioned previously, it will be seen that none of the three coarse aggregates were totally at the satisfactory level, and further that the friction

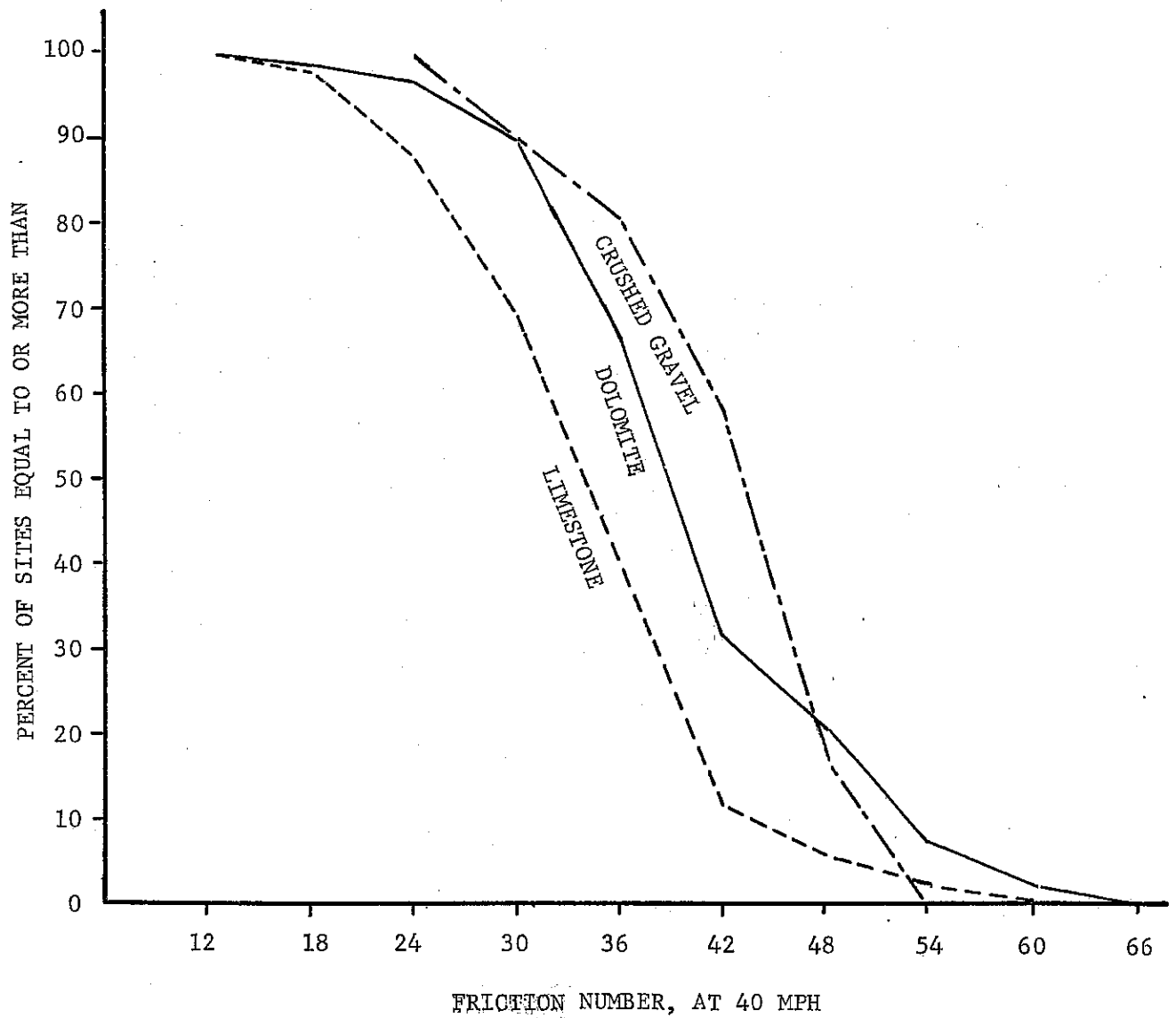


Figure 1. Cumulative frequency curves for friction quality of Class I bituminous concrete surfaces by type of coarse aggregate (6).

quality of the crushed limestone was well below that of the other two aggregates.

The maps of Figures 2 and 3 were constructed to pursue the matter further. In these maps, the State is divided into three areas in which the dividing lines separate the northern, central, and southern Districts. Results of classifying the friction numbers for the test sites in the three established friction levels for each of the three areas are shown in Figure 2. The high proportion of the tested Class I bituminous concrete surfaces in the southern Districts showing FN values in the lower quality levels as compared with the pavements of the northern and central districts is readily observable. Referring to Figure 3 on which are shown the locations of principal available coarse aggregate sources, and considering that aggregates ordinarily are shipped to construction projects from the nearest sources, the association of limestone coarse aggregate with generally lower quality friction in Class I surfaces is apparent.

With the knowledge that friction quality very probably will be reduced by traffic action, and that the reduction will be a function of the coarse aggregate in Class I bituminous concrete, wear curves were developed from the available data for various conditions, as shown in Figures 4, 5, 6, and 7. Except for the crushed gravel wear curves that probably were greatly affected by the smallness of the available sample, the wear curves show the existence of degradation of frictional quality with accumulating traffic passage, as reported previously by others.

The effect that wear by studded tires, in use at the time of the study but now banned, had on friction quality was not determinable from the data.

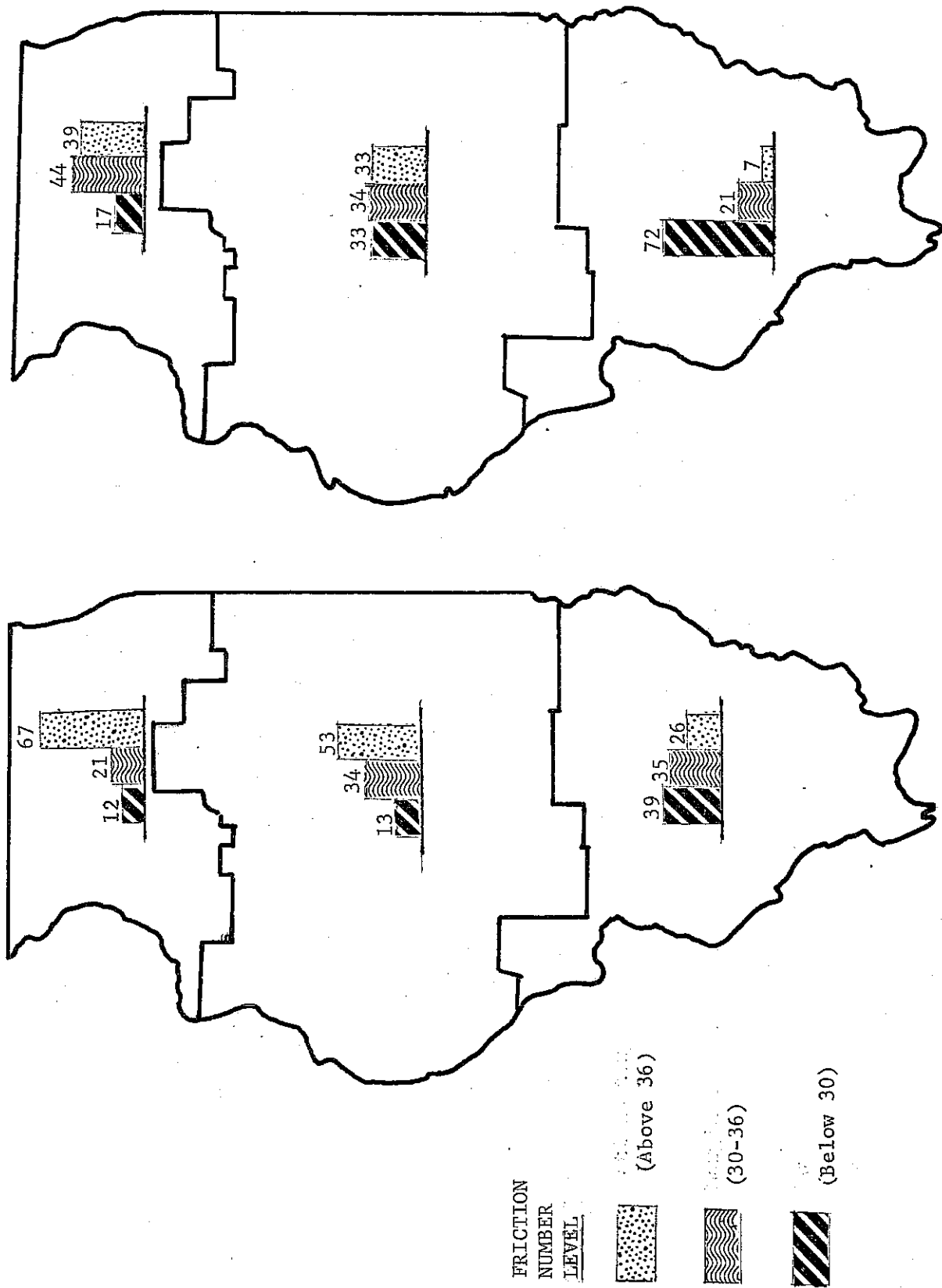


Figure 2. Friction-quality rating of Class I Bituminous concrete surfaces by geographical areas (6).

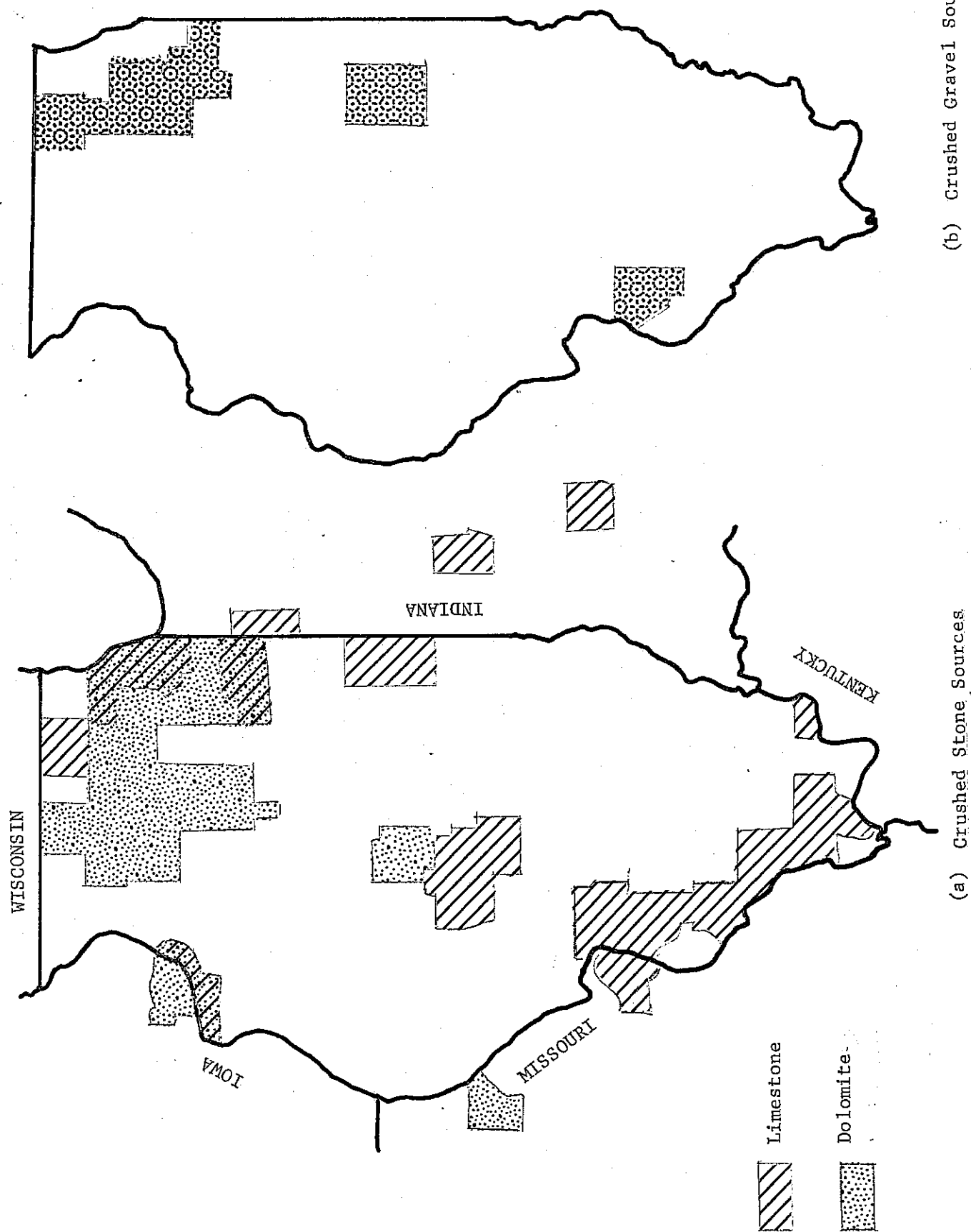


Figure 3. Map showing geographical source of coarse aggregate for Class I bituminous concrete surfaces (6).

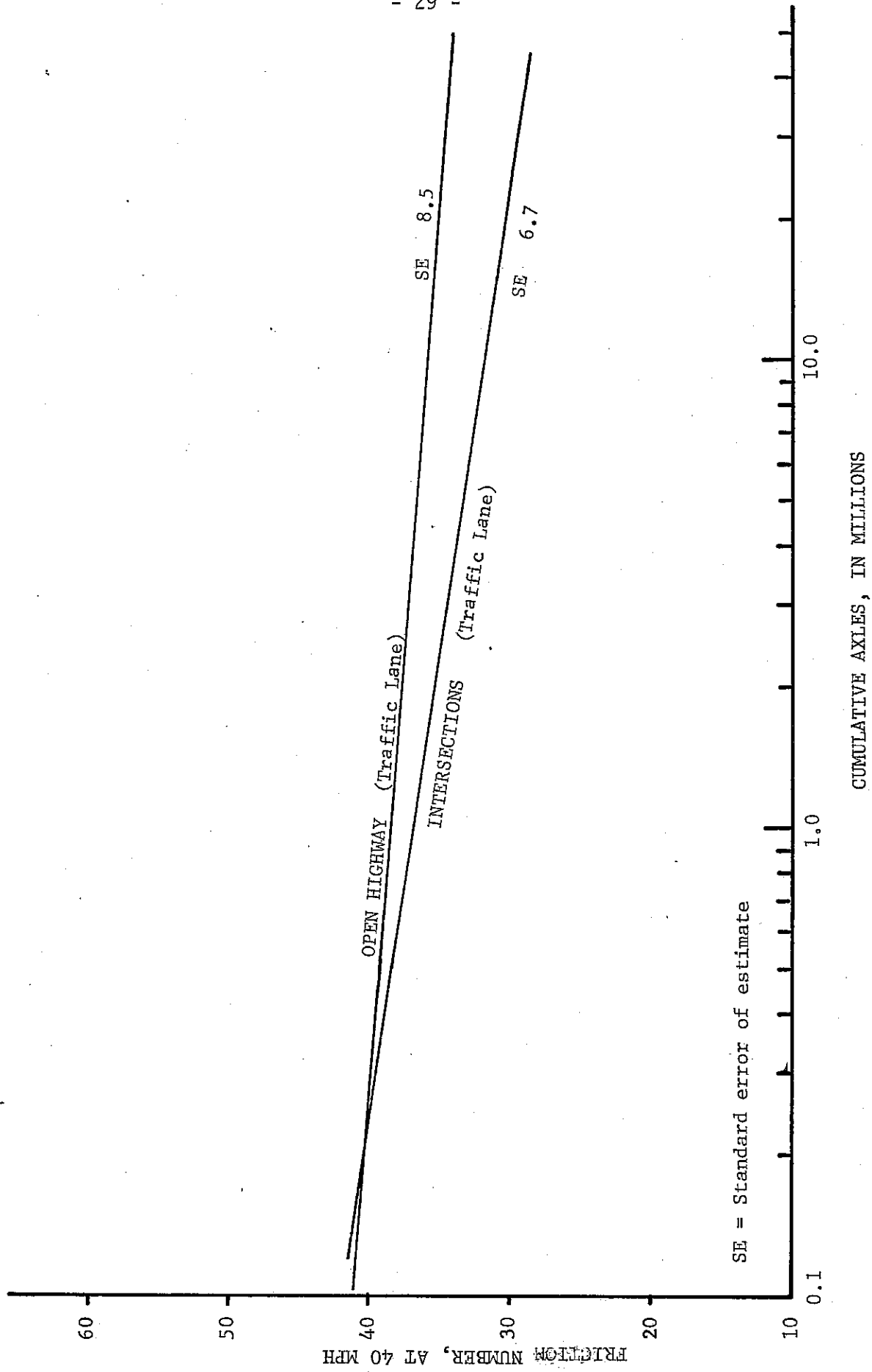


Figure 4. Wear curves for Class I bituminous concrete surfaces by location (6).

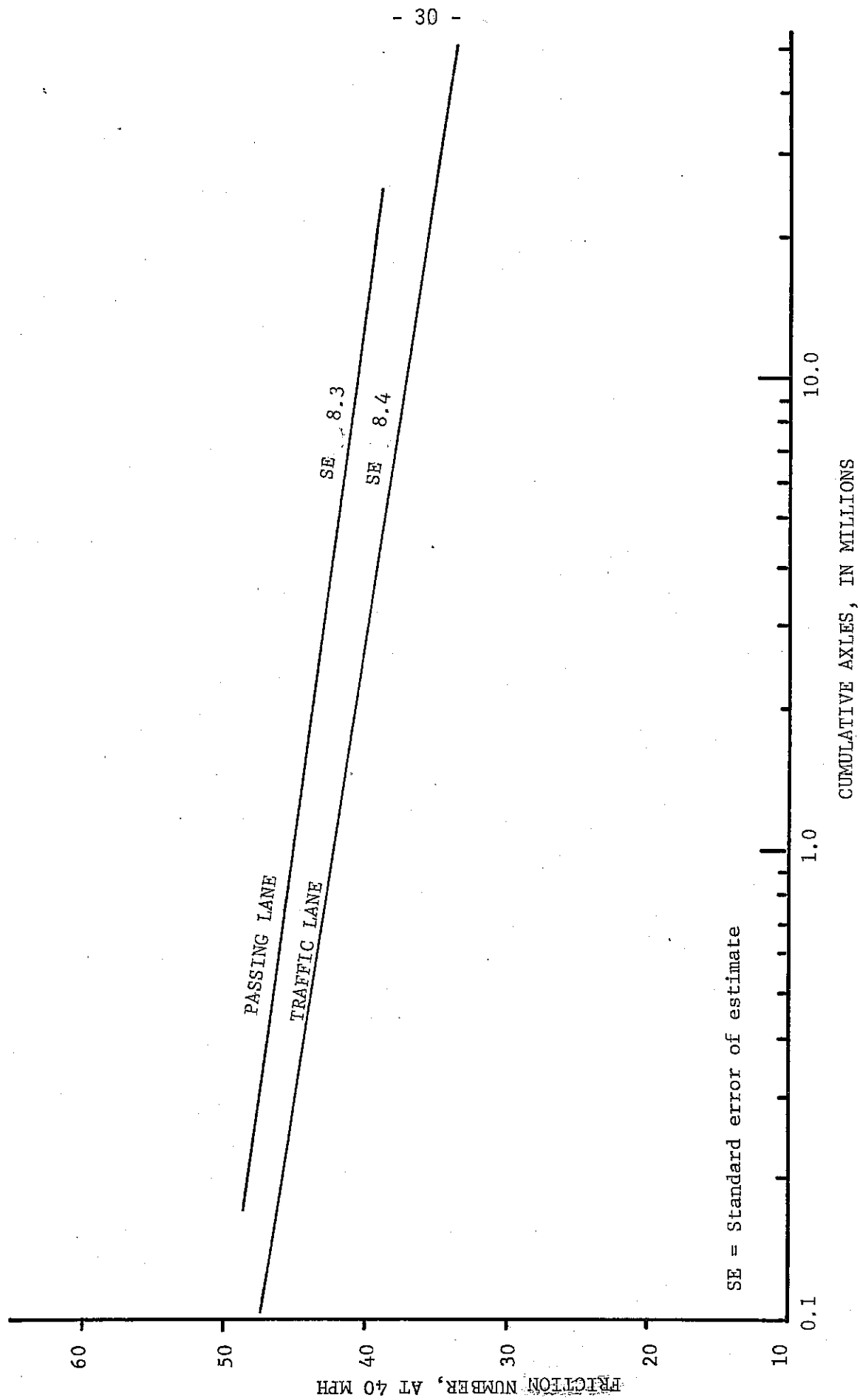


Figure 5. Wear curves for traffic and passing lanes of Class I bituminous concrete surfaces (6).

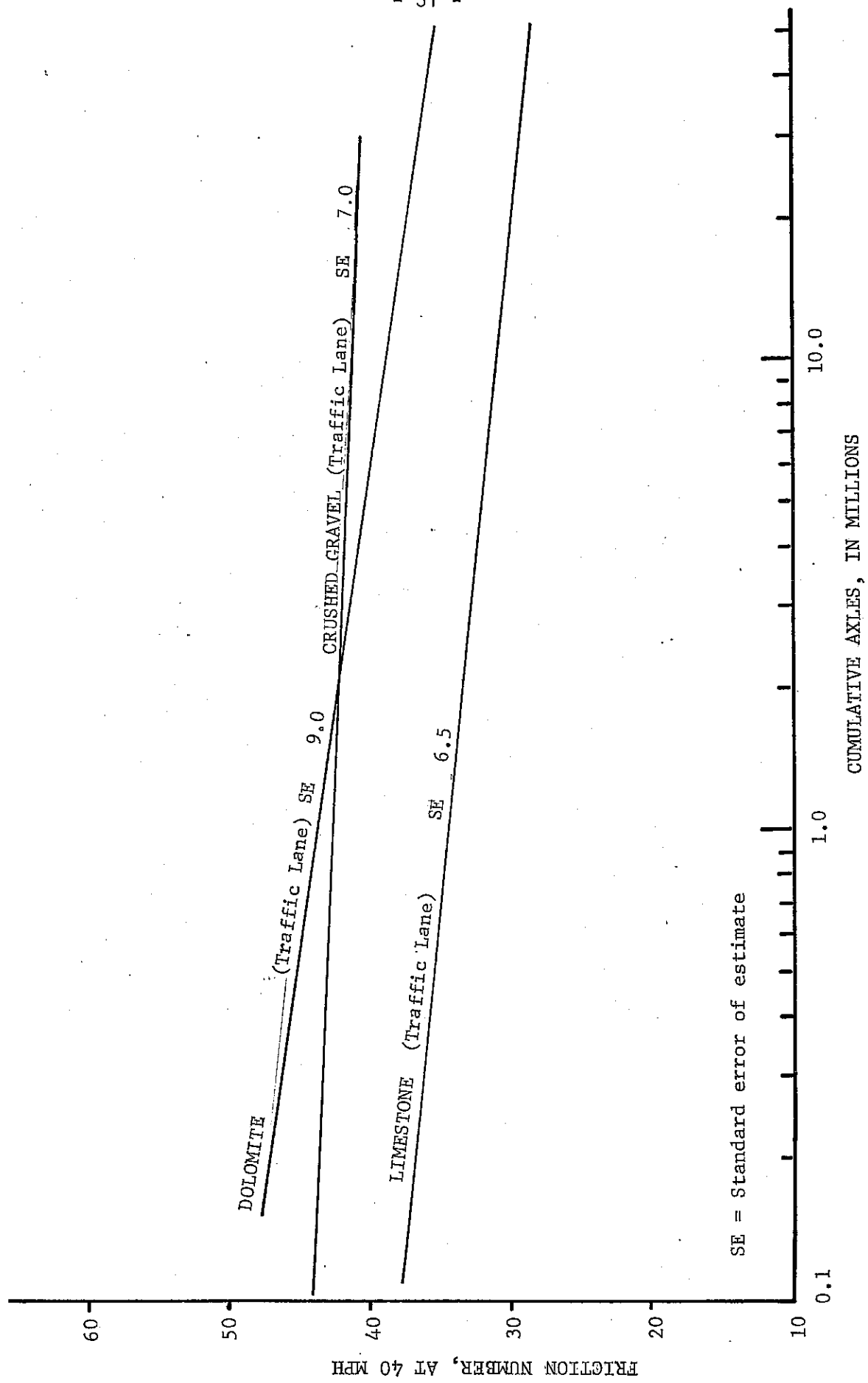


Figure 6. Wear curves for Class I Bituminous concrete surfaces on open highway by type of coarse aggregate (6).

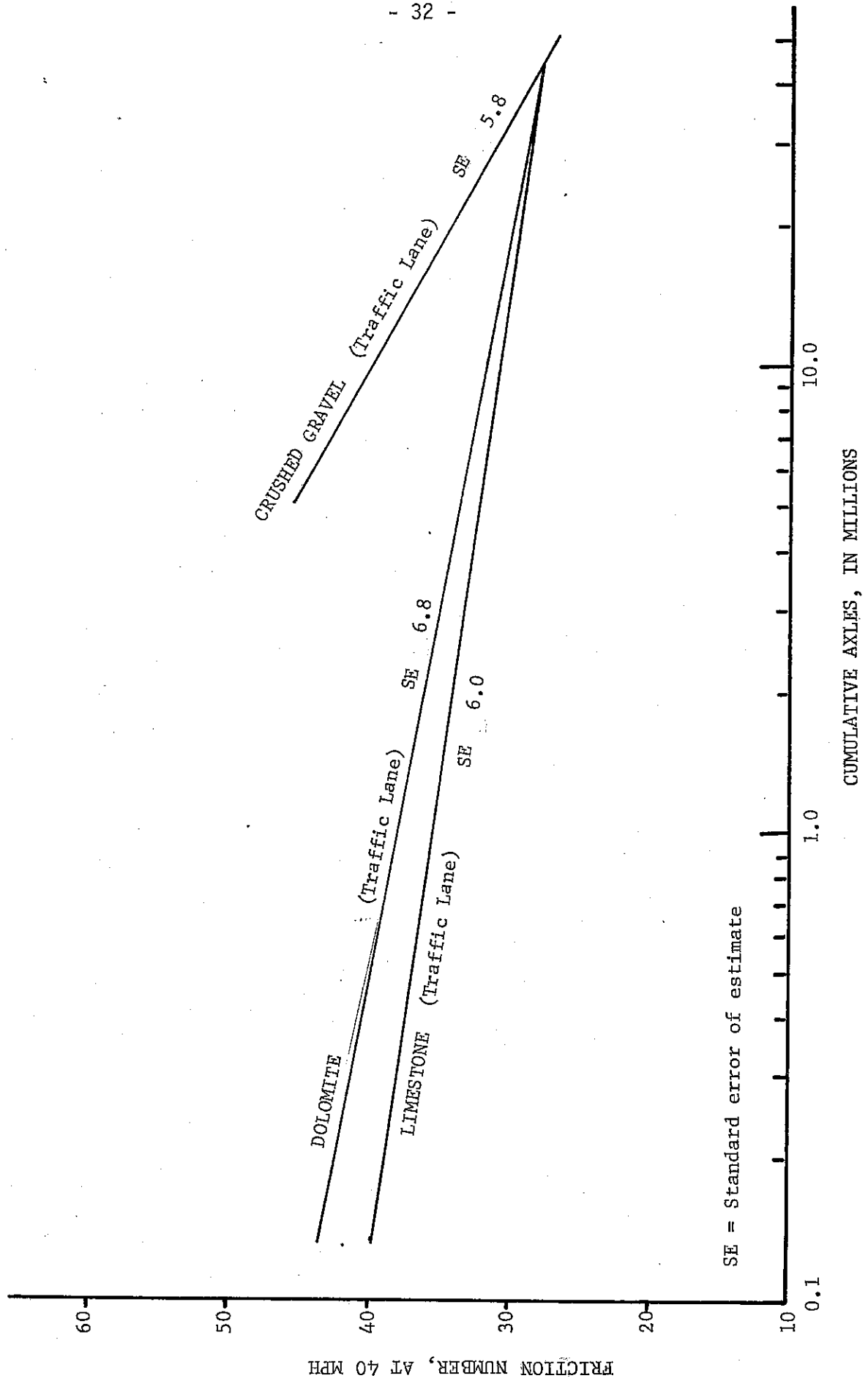


Figure 7. Wear curves for Class I bituminous concrete at intersections by type of coarse aggregate (6).

PORTLAND CEMENT CONCRETE PAVEMENT

Portland cement concrete (PCC) pavements in Illinois are constructed to generally accepted standards of quality. All of the surfaces tested probably were textured by dragging two separate double thicknesses of burlap over the freshly placed concrete, a practice that has since been replaced by a more positive form of texturing as discussed in Chapter Three. Texturing, and physical characteristics of the fine-aggregate fraction that is normally the principal component of the surface layer of pavement concrete, generally are considered to have the greatest effect on the frictional quality of PCC surfaces (3)(4). Unlike bituminous surfaces, the coarse aggregate fraction will be exposed only by abnormal or extremely prolonged wear.

The PCC friction measurement data were analyzed in a manner similar to that used for the Class I bituminous concrete data. Using the same three geographic areas that were employed in analyzing the bituminous concrete data and grouping the data in the three levels of friction quality established for the study, the results shown in Figure 8 were achieved. It will be noted that very small percentages of the test sites were at the low level in central and southern Illinois, with moderate percentages prevailing in northern Illinois. Much higher traffic volumes and greater use of the now-banned studded tires are believed to be the most likely causes of the lower values in the northern section. An analysis of the influence of the sand component of the PCC divided between subangular and rounded particles showed no distinguishable differences.

As was done for the Class I bituminous concrete, an analysis was made of the effect of traffic use on FN values with the results shown in

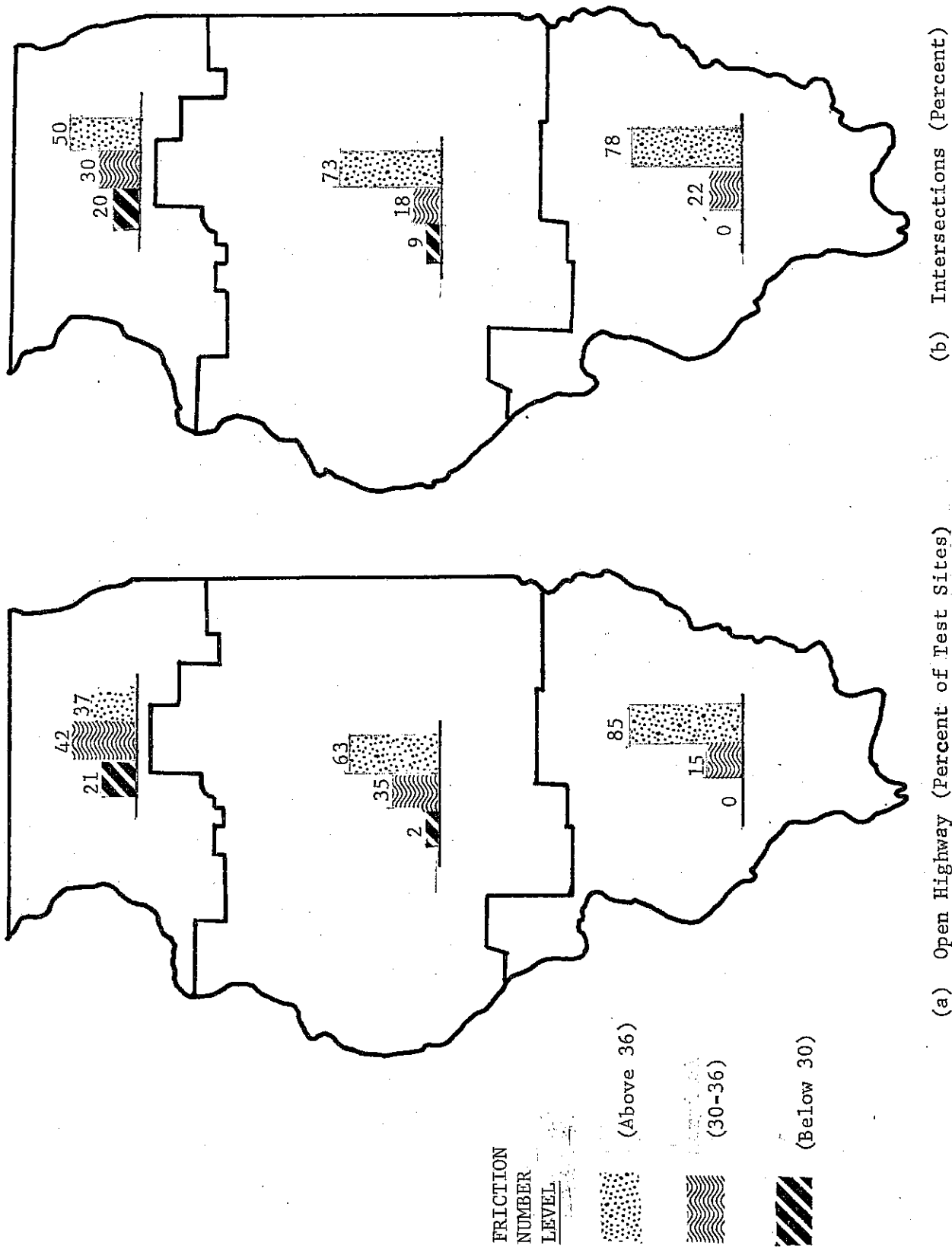


Figure 8. Friction-quality rating of PCC surfaces by geographical area (6).

Figures 9 and 10. Predictably, wear is shown to have an adverse effect on friction quality.

The effect of studded tire wear on the friction quality of PCC pavement surfaces could not be quantified.

FRICTION LIFE OF CLASS I AND PCC PAVEMENTS

Available friction measurement data for new pavements suggest that the as-constructed friction numbers for both PCC and Class I bituminous concrete pavements can be expected to be at or above the minimum value ($FN_{40}=37$) established for the satisfactory friction level. However, as mentioned previously and as demonstrated in Figures 11, 12, and 13, the friction quality of both PCC and Class I bituminous concrete surfaces did not always remain at the satisfactory level. Although some inconsistencies in the plotted information can be observed, probably due to the lack of sufficient data, it will be noted that the performance of the Class I bituminous concretes containing limestone coarse aggregate is significantly below that of the PCC regardless of coarse aggregate type and that of the Class I bituminous concretes containing other coarse aggregates.

Using the wear curves heretofore presented and converting the cumulative axle applications to traffic volumes using Statewide average distributions of vehicle type and lane usage, Table 2 was developed to suggest the anticipated friction life that might be expected for PCC and Class I bituminous concrete pavement surfaces constructed as those of the study on the Interstate-primary systems in Illinois.

Looking first at the PCC side of the table, the friction-life figures will be seen to suggest that PCC highways in southern Illinois

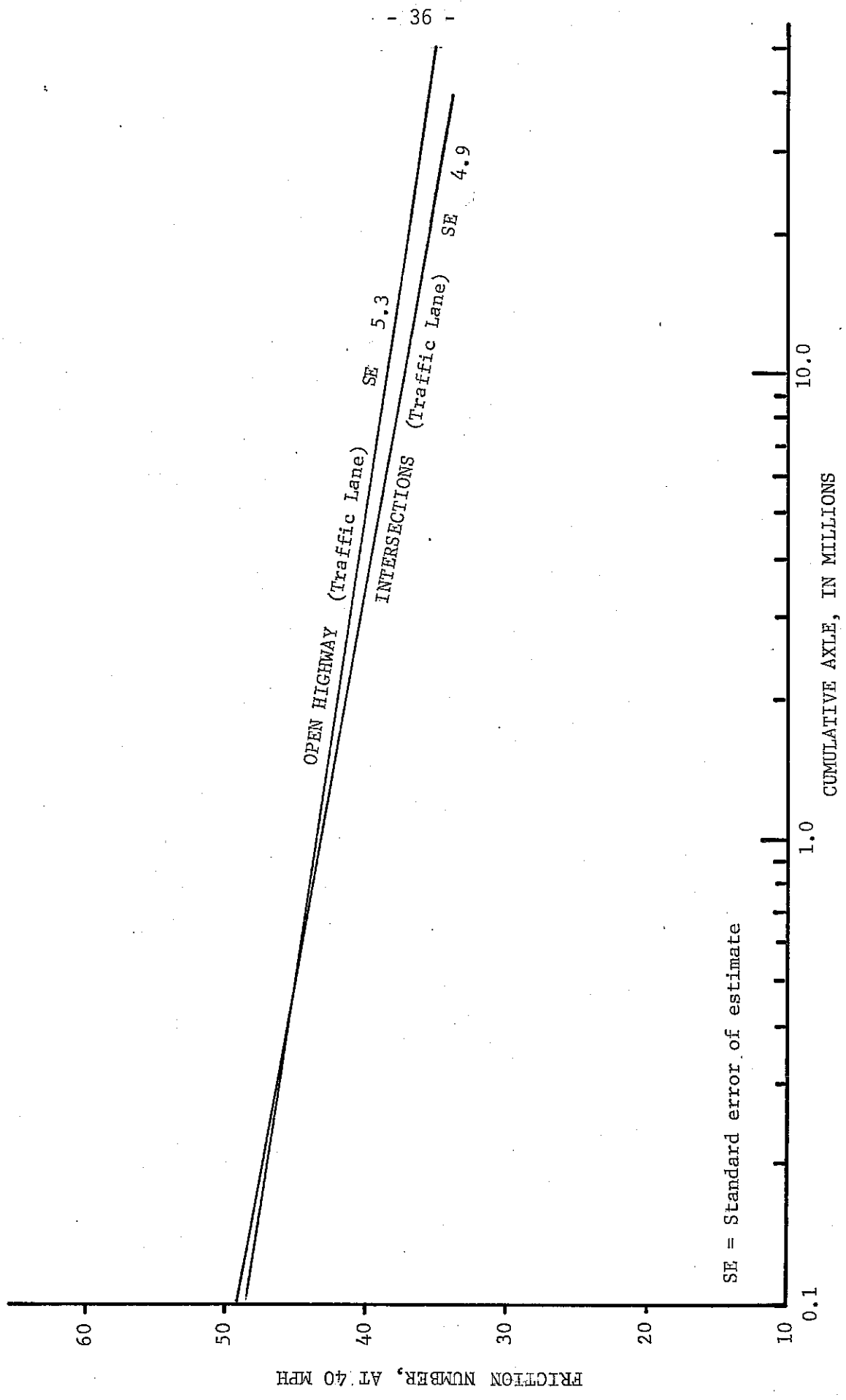


Figure 9. Wear curves for PCC surfaces by location (6).

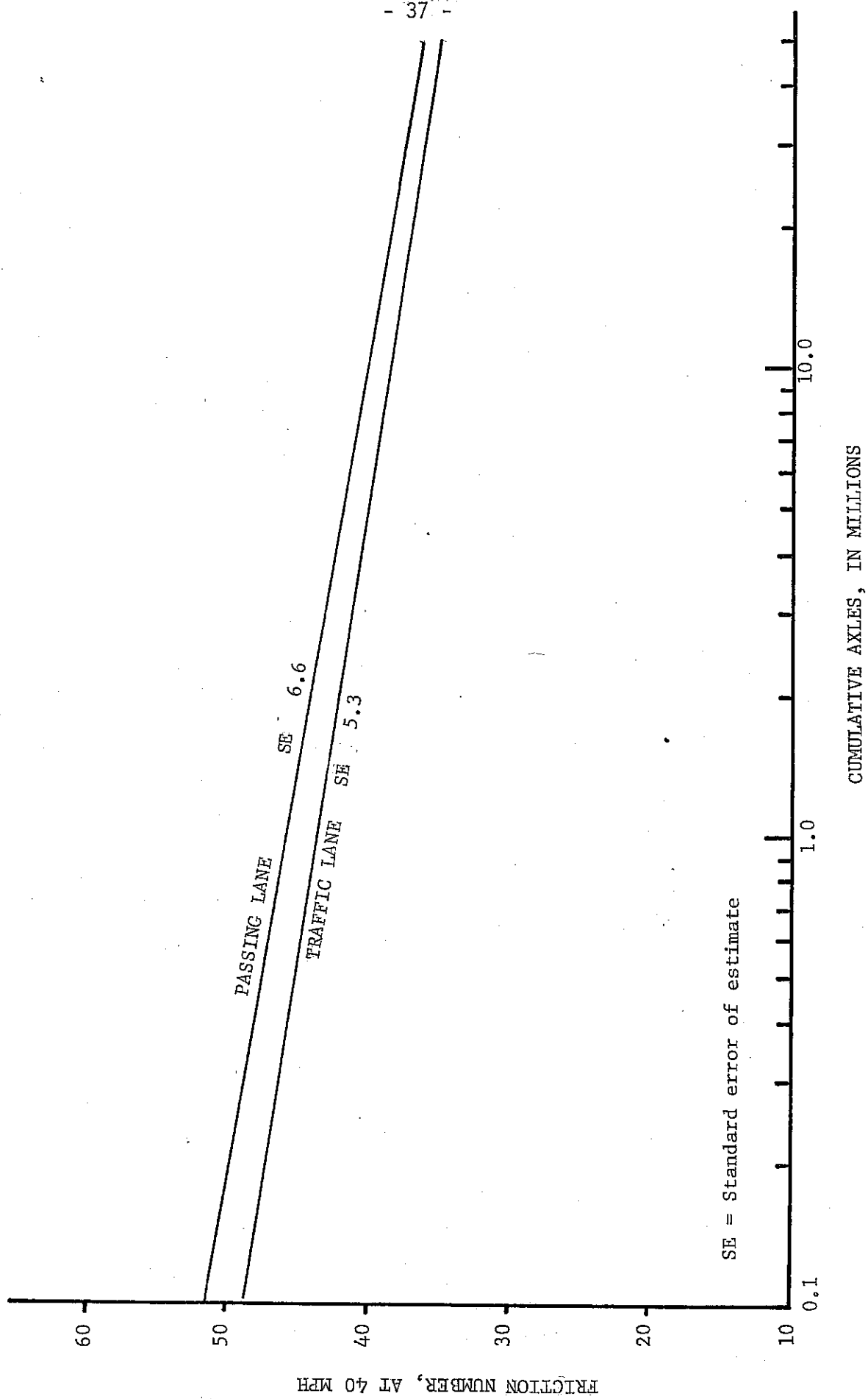


Figure 10. Wear curves for traffic and passing lanes of PCC surfaces (6).

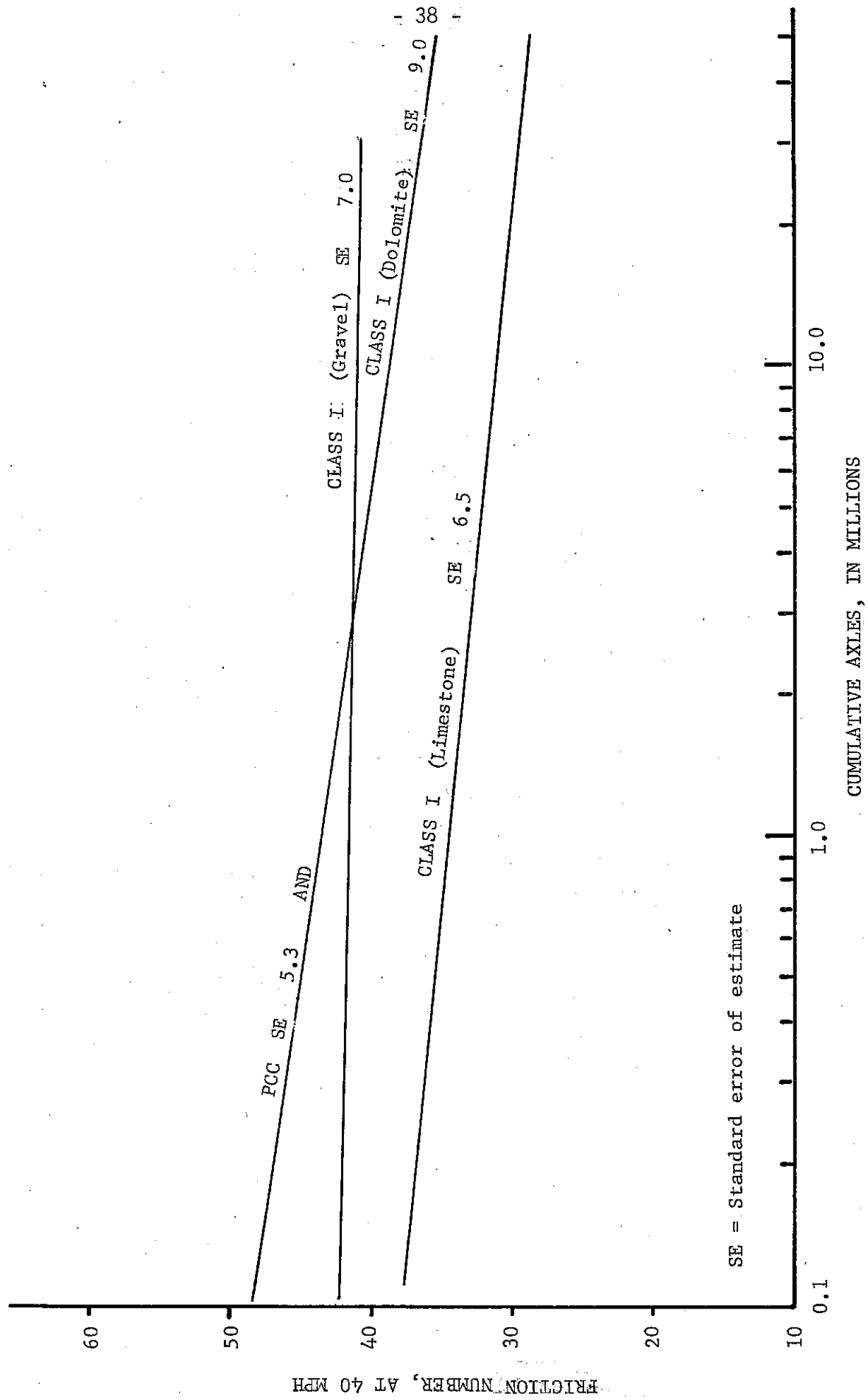


Figure 11. Comparison of wear curves for traffic lanes on open highway by type of surface (6).

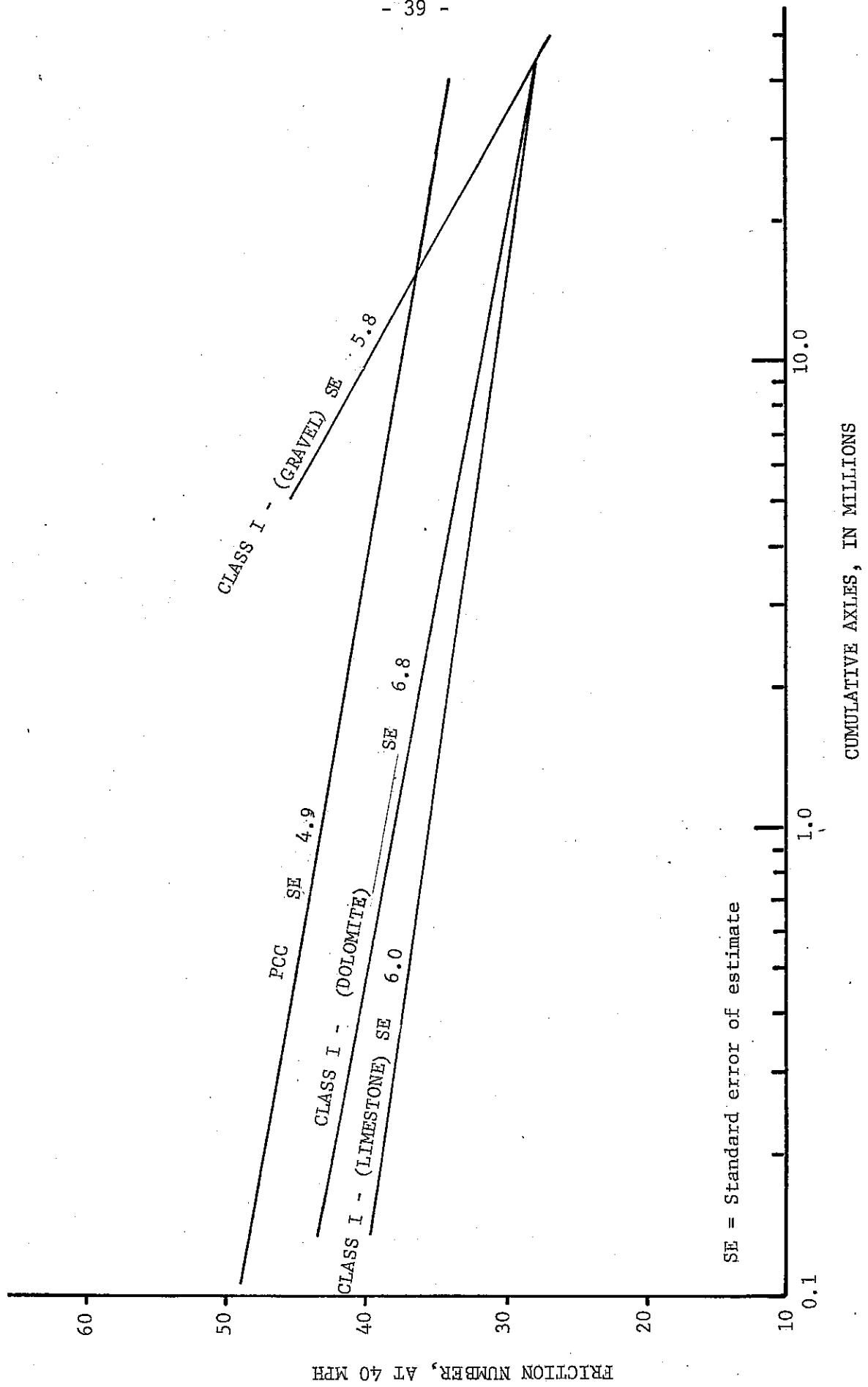


Figure 12. Comparison of wear curves for traffic lanes at intersections by type of surfaces (6).

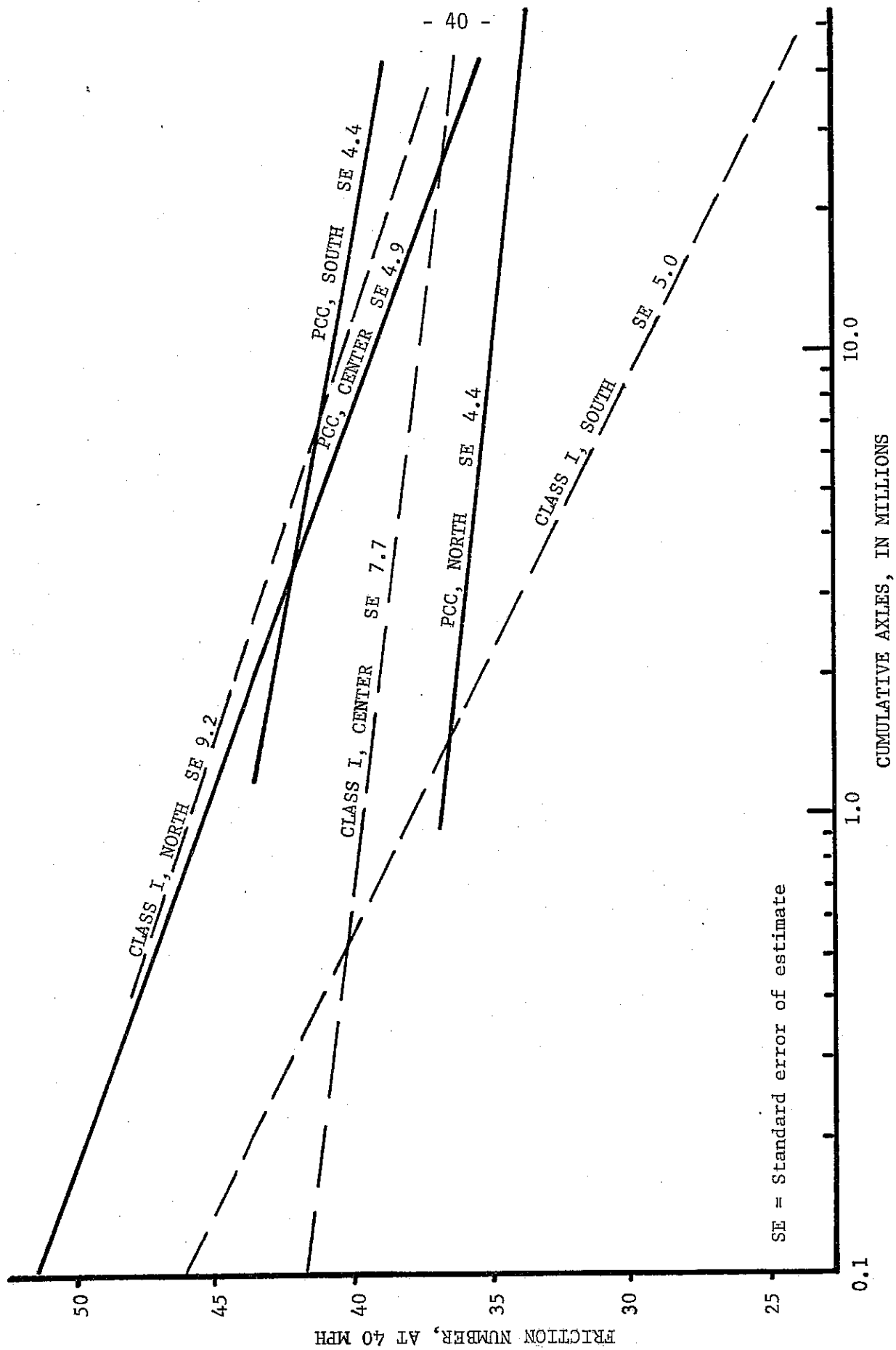


Figure 13. Wear Curves for PCC and Class I surfaces by geographical area (6).

TABLE 2 (6)

ANTICIPATED FRICTION LIFE OF INTERSTATE AND OTHER PRIMARY HIGHWAYS TO AN FN_{40} OF 35^{1/}

ADT	Mean Friction Life, In Years					
	PCC			Class I		
	North	Central	South	North	Central	South
<u>Two-Lane Highways</u>						
1,000	18	20+	20+	15+	15+	5
2,500	7	20+	20+	15+	15+	2
3,000	6	20+	20+	15+	15+	1
4,000	5	20+	20+	15+	15+	1
5,000	4	20+	20+	15+	15+	1
6,000	3	19	20+	15+	15+	1
<u>Four-Lane Highways</u>						
5,000	4	20+	20+	15+	15+	2
10,000	2	12	20+	15+	15+	1
15,000	1	9	20+	15	15+	0
20,000	1	6	20+	12	15+	0
25,000	0	5	20+	9	15+	0
30,000	0	4	20+	8	15+	0
35,000	0	4	20+	7	15	0
40,000	0	3	20+	6	14	0
<u>Six-Lane Highways</u>						
20,000	2	15	20+	15+	15+	1
25,000	2	12	20+	15+	15+	0
30,000	1	10	20+	15+	15+	0
40,000	1	8	20+	14	15+	0
50,000	1	6	20+	11	15	0
60,000	0	5	20+	9	13	0
80,000	0	4	20+	7	10	0
100,000	0	3	20+	6	8	0

^{1/}Derived from measurements made in 1969-71 on existing pavements.

and most two-lane PCC highways in central Illinois such as those in the study can be expected, on the average, to retain a satisfactory level of friction based on the levels established for the study for most of their 20-year structural design life. The figures suggest further that PCC pavements of multi-lane highways of central Illinois, with one exception, and all of the highways in northern Illinois such as those of the study, on the average, can be expected to have a friction life below, and often well below, the structural design life of 20 years.

As for the Class I bituminous concrete surfaces, the friction-life figures will be seen to suggest that those of northern and central Illinois, except when on the most heavily traveled expressways should, on the average, retain satisfactory friction through an assumed 15-year structural design life. The friction-life figures further suggest extremely short lives, on the average, for the Class I bituminous concrete pavements of southern Illinois where the limestone coarse aggregate now known to be lacking in frictional resistance was used.

Although the information in Table 2 is believed to have some very general application in situations where conditions known to affect skid resistance have remained substantially the same as when the reported friction measurements were made in 1969-71, it is important to consider that some very important changes, all of which can be expected to have an advantageous effect on friction quality, have been made. These include:

- (1) A banning in 1976 of studded-tire use, a move that was particularly beneficial to friction quality in northern Illinois where 13 percent of the passenger cars had been equipped with studded tires by 1972.

- (2) A limitation placed in 1975 on the use of limestone coarse aggregate in Class I bituminous concrete surface course to secondary roads and streets having design ADT volumes of 500 and less, and to use only in a 50-50 blend with slag coarse aggregate elsewhere other than on the most heavily traveled multilane expressways in the Chicago Metropolitan Area where no limestone coarse aggregate is now used.
- (3) Elimination in 1976 of the burlap drag system of PCC pavement texturing and introduction of a much more positive system of texturing that includes application of an artificial turf drag longitudinally followed immediately by transverse tining.

The design and construction changes of items 2 and 3 preceding are treated more fully in Chapter 3 following.

CLASS B BITUMINOUS CONCRETE PAVEMENT

Class B bituminous concrete is used mainly as a surface course overlying a flexible base on secondary roads and contains a softer asphalt component than Class I mixtures that are designed for application on rigid bases. Also, a larger nominal top size of aggregate (1 in.) is permitted and aggregate controls generally are less stringent.

Because of the relatively small number of test sites recommended by the Districts and a spotty geographic distribution within the State, it was concluded that an analysis of the detail applied to the sites on the Interstate-primary system would not be productive. Therefore, effort was limited to the following two analyses: (1) site distribution by coarse aggregate type (crushed stone and gravel) within the three levels of friction quality established for the study; and (2) the effect of

pavement wear on friction quality. Distribution of the test sites according to friction quality level is shown in Table 3. It will be noted that well over two thirds of the sites on the open highway and about two thirds of the sites at intersections were at the satisfactory level of friction quality, with the remainder distributed between the marginal and low levels. Those having gravel coarse aggregate in the bituminous mixtures showed generally better skid resistance than did those with crushed stone coarse aggregate. Wear curves developed from the data are shown in Figure 14. Here again, the surfaces containing the gravel aggregates are indicated to be the better performers, showing lower rates of wear than did the surfaces with crushed stone coarse aggregate. Aggregate data were combined to provide the wear curves of Figure 15.

In general, the analyses of the Class B bituminous concrete data appear to show trends similar to those shown in the Class I bituminous concrete data analyses. However, neither the need for change nor the number and distribution of the sites tested has been considered to be sufficient to warrant general design or construction changes that almost certainly would add substantially to the cost of this type of pavement.

CLASS A BITUMINOUS SURFACE TREATMENTS

Bituminous Surface Treatments in Illinois are comprised of one or more layers of asphalt and seal coat aggregate. The performance of these surface treatments, including their skid resistances, generally is acknowledged to be extremely sensitive to construction procedures that, if improperly executed, produce low and even dangerous levels of skid resistance. The sensitivity was reflected in the study by wide variations in the measured values of friction, both among test sites and within individual sites.

TABLE 3

FRICTION-LIFE RATING OF CLASS B BITUMINOUS CONCRETE SURFACES
(Adapted from (6))

Location	Aggregate Type	Percent of Total Sites with FN			Total
		Below 30	30-36	Above 36	
Open highway	Gravel	-	-	42(11)	42(11)
	Crushed Stone	12(3)*	4(1)	42(11)	58(15)
	All	12(3)	4(1)	84(22)	100(26)
Intersection	Gravel	-	7(1)	37(5)	43(6)
	Crushed Stone	21(3)	7(1)	28(4)	57(8)
	All	21(3)	14(2)	65(9)	100(14)

*Numbers in parentheses are numbers of test sites

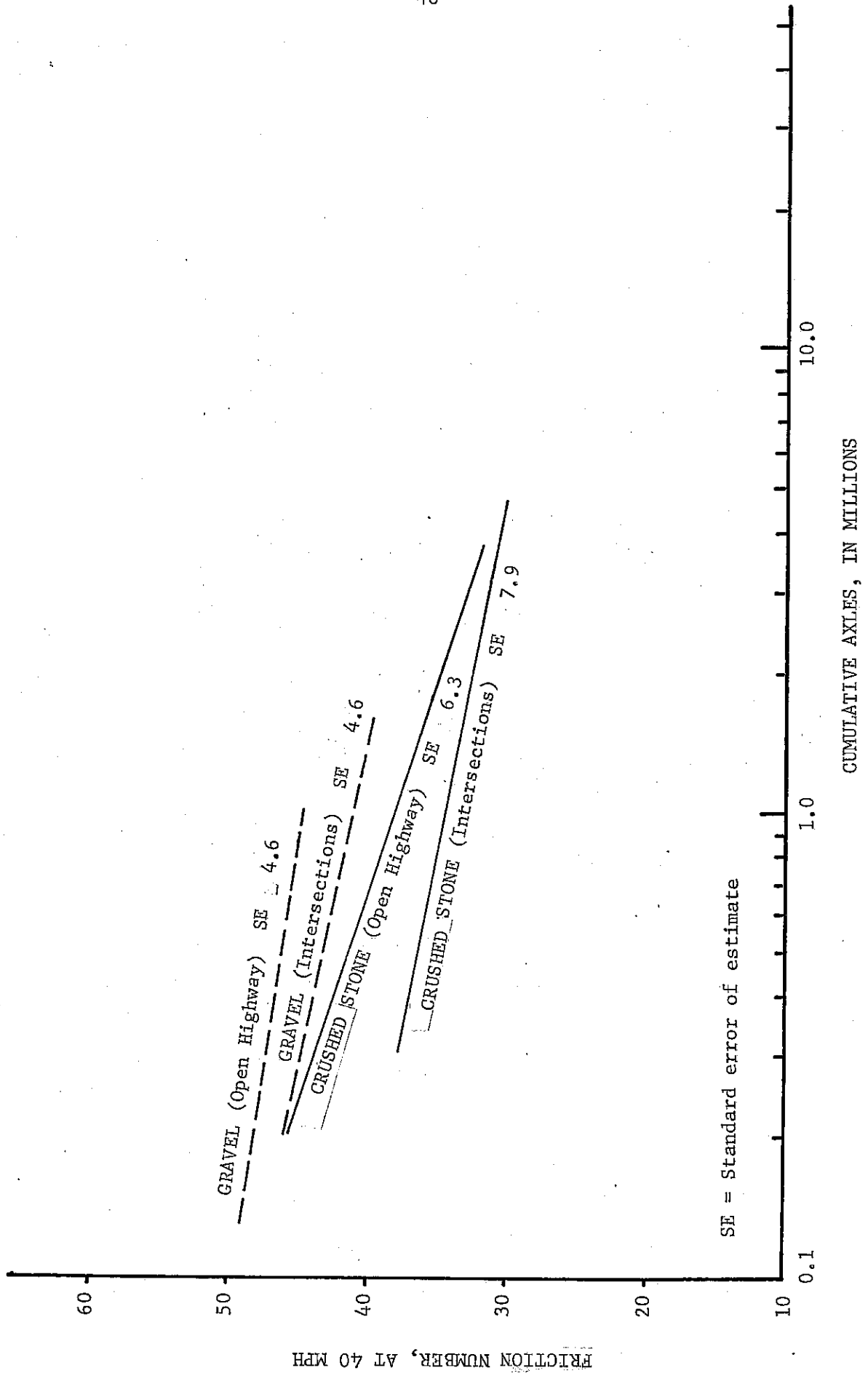


Figure 14. Wear curves for Class B Bituminous concrete surfaces by location and by aggregate type (6).

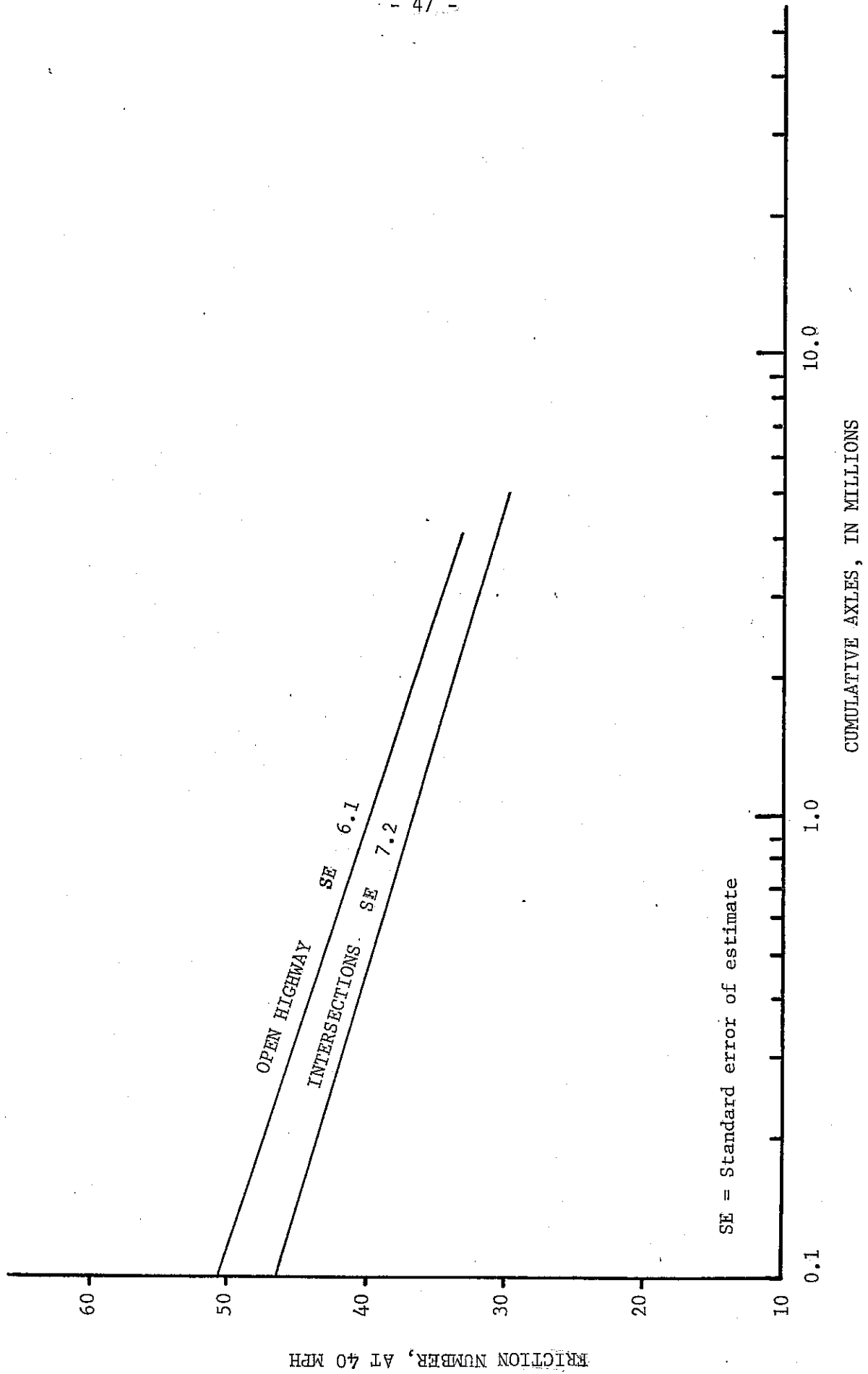


Figure 15. Wear curves for Class B bituminous concrete surfaces by location (6).

The analyses to which the surface treatment data were subjected were the same as those for the Class B bituminous concrete. Distribution of the test sites by seal coat aggregate (crushed stone and gravel) within the three levels of friction quality established for the study are shown in Table 4. A relatively high percentage of sites, especially sites with crushed stone seal coat aggregate, will be seen to exist at the low level. An analysis of the effect of age on frictional characteristics showed no trends. Wear curves developed for the surface treatments are shown in Figure 16. A substantial loss of friction quality through traffic wear is indicated.

As with the Class B bituminous concrete, the results of the Class A surface treatment study are not considered to provide sufficient support for any general design or construction changes at this time.

FRICITION LIFE OF CLASS B AND CLASS A SURFACES

As was done for the PCC and Class I bituminous concrete pavements, the wear curves of Figures 15 and 16 for the Class B bituminous concrete and the Class A surface treatments were used in conjunction with cumulative axle applications converted to traffic volumes based on county highway vehicle classification counts to develop the friction-life values shown in Table 5. The data for the Class B surfaces will be seen to suggest that, on the average, they can be expected to maintain acceptable friction quality throughout most of a 15-year structural design life when the ADT does not exceed 500. This will cover a major portion (80 percent) of the mileage of Class B surfaces on secondary and local roads in Illinois. The data for the Class A bituminous surface treatments will be seen to suggest considerably shorter average friction lives.

TABLE 4 (6)

FRICTION-QUALITY RATING OF CLASS A BITUMINOUS SURFACE TREATMENTS^{1/}

Location	Aggregate Type	Percent of Total Sites with FN			
		Below 30	30-36	Above 36	Total
Open highway	Gravel	9 (4)*	11 (5)	25 (11)	45 (20)
	Crushed stone	24 (11)	11 (5)	20 (9)	55 (25)
	All	33 (15)	22 (10)	45 (20)	100 (45)
Intersection	Gravel	20 (6)	10 (3)	17 (5)	47 (14)
	Crushed stone	36 (11)	10 (3)	7 (2)	53 (16)
	All	56 (17)	20 (6)	24 (7)	100 (30)

*Numbers in parentheses are numbers of test sites.

^{1/} Derived from measurements made in 1969-71 on existing pavements

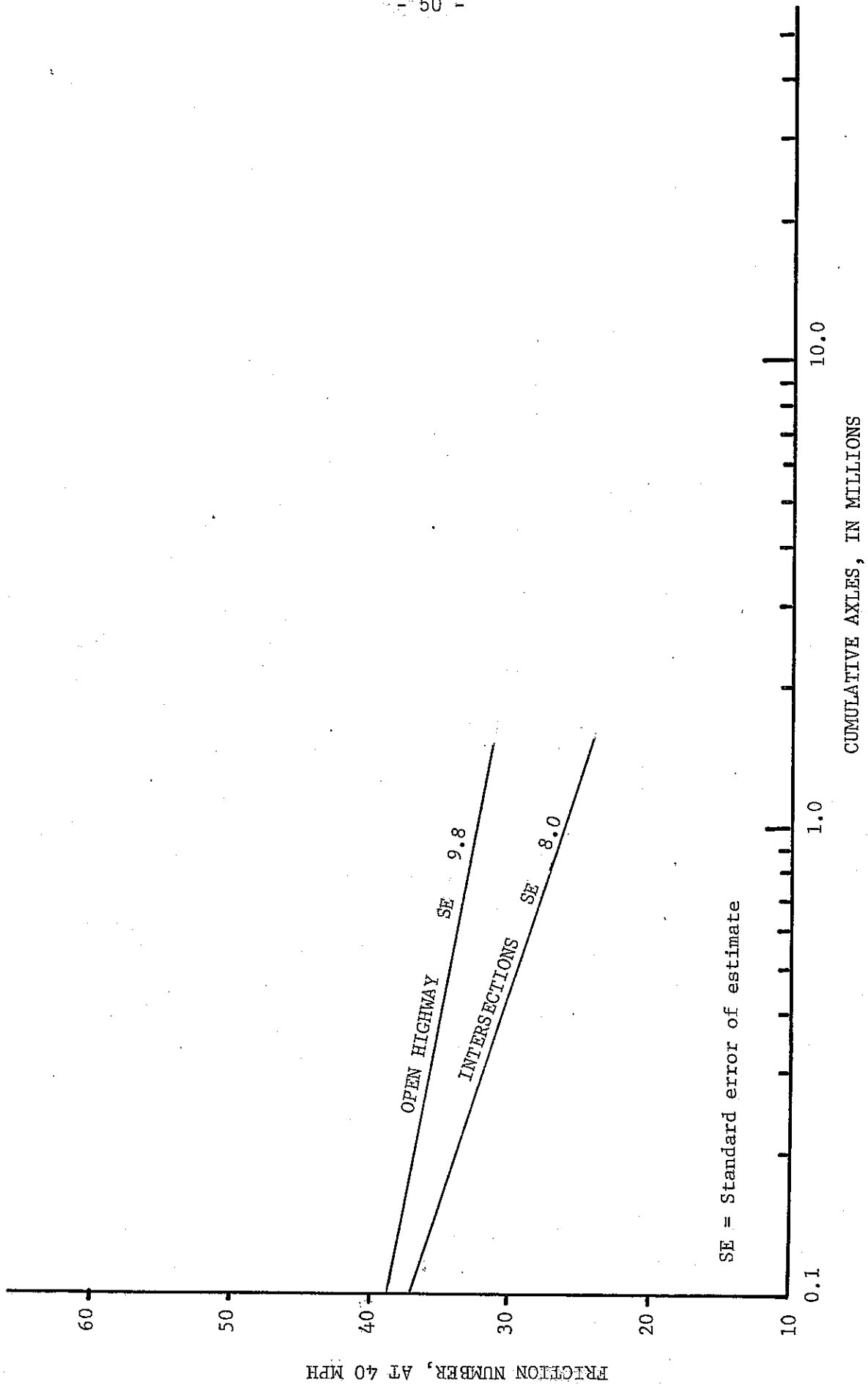


Figure 16. Wear curves for Class A bituminous surface treatments by location (6).

TABLE 5 (6)

ANTICIPATED FRICTION LIFE OF COUNTY HIGHWAYS AT FN_{40} of 35^{1/}

ADT	Mean Skid Life, in Years	
	Class B	Class A
250	15+	4
500	14	2
750	10	1
1000	7	1
1250	6	-
1500	5	-

^{1/}Derived from measurements made in 1969-71 on existing pavements

These lives, however, probably are not very far different from the average service life of this type of surface.

CONTINUATION OF SAMPLING AND ANALYSIS

Following the completion of the Statewide sampling of the friction quality of Illinois pavements in 1969-71 and the development of a fairly broad data base reflecting conditions at the time of the survey, sampling and analysis have continued, but on a more selective basis. The data base has been expanded each year through friction testing at from 125 to 150 high-skid accident sites, limited check-testing of new construction, and inventory-type testing on selected main arteries of travel including the Chicago Stevenson, Calumet, Dan Ryan, Eisenhower and Kennedy Expressways, and Interstate 80 across north-central Illinois. Some of this work has produced information that has been applied in pavement design, construction and maintenance and is discussed further in Chapter 3. Reference is also made to a portion of the work in Chapter 4.

A considerable amount of effort also has gone into preliminary developmental evaluations involving wet-pavement time and wet-pavement accidents.

In many respects, much of the friction sampling and analytical work that Illinois has done since the conclusion of the initial survey of 1969-71 and up to the present has been preparatory to the development of what is intended to be a sound, viable and comprehensive program of sampling and analyzing the frictional characteristics of selected roadway sections that can assist management for many years to come with a minimum of adjustment in making the most effective use of available

resources in a skid-accident reduction program. Illinois is approaching development of the program with considerable caution to obtain every possible assurance that the program can respond properly to needs and yet remain within practical funding and manpower constraints.

Current work, and that planned for the immediate future, is under Illinois project IHR-407, "Pavement Friction Inventory."

IN THE FUTURE

Preparatory work leading to the development of a detailed work plan for sampling and analysis of representative roadway sections is nearing completion and development of the work plan is in process. Careful attention will be paid to pertinent portions of FHWA Technical Advisory T 5040.17, dated December 23, 1980 (appended hereto as Appendix A).

CHAPTER THREE

ASSURING ADEQUATE SKID RESISTANCE THROUGH PAVEMENT DESIGN, CONSTRUCTION, AND MAINTENANCE

The sampling survey of Illinois pavements reported in Chapter 2, and especially that part of the survey conducted in 1969-71, was designed to furnish, and has furnished, both a general overview of the frictional characteristics of the highway system as a whole in Illinois and information on the magnitude and durability of the friction offered by the various surface types, components, and operating conditions existing at the time of the survey. For the great majority of pavements in service today, and for a limited mileage continuing to be built (for example, Class I Mixture C and many Mixture D surface courses), the results, although not always as definitive as current knowledge suggests to be desirable, offer a good representation of the frictional properties to be expected.

As a result of the sampling work that related to specific surface types, and through application of new knowledge gained through experimentation by Illinois and other agencies, older pavement design, construction, and restoration practices that were found to produce less-than-desirable levels of friction have been replaced largely by better practices. The changes in practice that have been made since the survey for the specific purpose of improving frictional quality are summarized in the first section of Chapter 3. Detailed information concerning the changes is available in Illinois Department of Transportation policy statements, standard specifications, and special provisions.

Experimental work that has been done in Illinois to improve friction quality, and evaluative studies that have been made through monitoring of the effectiveness of changes in practice that have been introduced to improve skid resistance, are summarized in the second part of Chapter 3. Plans for future experimentation and evaluative studies are indicated. More detailed information on the experimentation and evaluation work is available in (7), (8), (9), (10), (11), and (14).

CHANGES IN PRACTICE

Changes in practice that have been introduced to upgrade the friction quality of newly constructed pavements, as summarized in this section of the report, include major revisions in coarse-aggregate use in Class I bituminous concrete surface course mixtures, introduction of the open-graded asphalt friction course, introduction of a plasticized bituminous hot-mix seal, and a change in the method of texturing portland cement concrete pavement surfaces.

CLASS I DENSE-GRADED BITUMINOUS CONCRETE SURFACE COURSE

As a result of the sampling survey finding that not all coarse aggregates were imparting desirable frictional characteristics to the Class I surface course mixtures (Standard Specification Mixture C) containing them under all service conditions, and making use of the results of ongoing experimentation, a number of changes were made in specifications covering coarse-aggregate use in these surfacings and in policies governing use of the aggregates to improve skid resistance in new construction.

The specification revisions consist mainly of the establishment of three surface course mixture classes, identified as Mixtures C, D, and E, where there was formerly only Mixture C. Under the new specifications, Mixture C remains unchanged, and Mixtures D and E provide increasingly greater restrictions on the types of coarse aggregate that can be used in them. Policies were then established regarding the traffic conditions under which each of the three mixtures is to be used.

Coarse aggregate types now permitted in the three Class I surface course mixtures are as follows:

<u>Coarse Aggregate</u>	<u>Mixture C</u>	<u>Mixture D</u>	<u>Mixture E</u>
Crushed slag	Yes	Yes	Yes
Crushed stone (a) Dolomite	Yes	Yes	Only when blended equally by volume with crushed slag
(b) Limestone	Yes	Only when blended equally by volume with crushed slag	No
Crushed gravel	Yes	Yes	Only when blended equally by volume with crushed slag
Chats	Yes	Yes	Only when blended equally by volume with crushed slag

Note: Coarse aggregate types are as defined in Standard Specifications.
The slag is air-cooled blast furnace slag.

Based on the sampling survey findings, the following policy was established to govern the use of the three Class I surfacing mixtures:

Mixture C - (1) On any road or street off the primary system
with a design ADT volume of 500 or less.

- Mixture D - (1) On any secondary or local road or street with a design ADT volume of over 500.
- (2) On any two-lane primary highway.
- (3) On any four-lane primary highway with a design ADT volume of 25,000 or less.
- (4) On any highway of six or more lanes with a design ADT volume of 60,000 or less.
- Mixture E - (1) On any four-lane highway with a design ADT volume of over 25,000.
- (2) On any highway of six or more lanes with a design ADT volume of over 60,000.

OPEN-GRADED ASPHALT FRICTION COURSE

In the search for more skid-resistant pavement surfaces, attention was directed several years ago toward open-graded asphalt paving mixtures that appeared to have an especially good potential for providing rapid rain-water drainage that could provide both improved wet-weather skid resistance at high speeds and minimize hydroplaning effects in heavy rainstorms. Immediate implementation on a large scale was slowed by difficulties sometimes encountered during construction and also during service performance. Many of the difficulties were attributed to the lack of an adequate mixture design procedure. A new design method developed and introduced experimentally by the FHWA in 1974 is believed to have provided the means for overcoming most of the former difficulties(13).

Designed particularly to make best use of its frictional properties and now known as an open-graded asphalt friction course (OGAFC), the OGAFC mixture consists predominantly of a narrowly graded coarse-aggregate fraction with a sufficiently high interstitial void capacity to provide for

a relatively high asphalt content, a high air-void content, and a relatively small fraction of fine aggregate. The coarse-aggregate fraction provides the desired structure and the fine-aggregate fraction acts as a filler to stabilize the coarse aggregate. The high asphalt content provides mixture durability and the high air-void content permits quick water drainage. A hard and durable coarse aggregate with high-quality friction characteristics is needed for good performance.

As the name indicates, the OGAFc is used primarily for its frictional quality rather than for its structural capabilities. A thin layer in the range of 5/8 in. to 3/4 in. normally is used. Thicknesses at the 3/4-in. level have been reported by Texas to provide better water drainage.

In addition to providing good surface friction and minimizing hydroplaning effects, the OGAFc is considered to offer other benefits, including improvement of riding quality on structurally sound surfaces, minimization of wet-weather splash and spray, lessening of wheelpath rutting, improved visibility of painted pavement markings, improved wet-weather night visibility, lower noise levels, and retardation of surface icing. There are limitations, too, including considerable sensitivity to mixture composition and to construction procedures, unfavorable reaction to cold-weather placing, and inferior performance at signalized intersections and on low-speed facilities with turning movements (tender mix).

The first OGAFc to be used in Illinois was placed experimentally in 1974. Through 1978, a total of 71 sections of OGAFc had been placed experimentally at 52 sites throughout the State. All of the work was done in general accord with FHWA recommendations. Summary details regarding the installations and subsequent performance evaluations are presented in the second section of this Chapter. Among the more significant observations

of performance to date are those suggesting poor structural behavior related to certain coarse aggregates available to Illinois.

Based on the results gained in the experimental work in Illinois, and on information that has become available from others, a Special Provision has been developed to govern future construction of open-graded asphalt friction courses in Illinois. The most recent issue of the Special Provision became effective January 2, 1980. This issue limits coarse aggregate use to crushed steel slag (slag from an open-hearth or basic-oxygen furnace), crushed air-cooled blast furnace slag, and crushed traprock (an igneous product of ore separation). A copy of the full Special Provision is included in this report as Appendix B.

PLASTICIZED BITUMEN HOT-MIX SEAL

Dense-graded Class I bituminous concrete surface courses with a maximum aggregate size between 3/8 in. and 1/2 in. have been found through the years to have good structural stability, durability, and usually acceptable frictional quality when the coarse aggregates have satisfactory frictional characteristics. These maximum aggregate sizes require constructed minimum layer thicknesses of about 3/4 in. and preferably somewhat greater. On many occasions, where the existing surface already meets, or very nearly meets, structural requirements, and the additional layer is needed mostly to improve riding quality or frictional quality, the use of a thinner layer can be economically advantageous.

With this economic advantage in mind, Illinois had done a considerable amount of experimentation in recent years with sand-asphalt mixtures, often generally patterned after the dense-graded bituminous concrete mixtures containing coarse aggregates, but with maximum size aggregates in the No. 4

sieve (3/16 in.) and 1/4-in. range. The experimentation has been concerned with both friction quality and structural stability.

One such mixture, which has been termed "plasticized bituminous hot-mix seal," has shown good stability and good frictional characteristics in many trial installations under a variety of conditions involving usually light-to-moderate traffic use, and its use is now permitted at the local road and street level where design ADT volumes are not in excess of 10,000. The mixture was developed in France and makes use of an additive that has been named Tapisable (presumably from the French "sable" meaning sand and "tapis" meaning carpet). The additive appears both to prevent stripping and improve workability.

The mixture, laid hot, consists of a paving asphalt (AC 100-120), the additive, and a blend of 70 percent natural sand and 30 percent stone screenings. No tack coat is needed.

The full specification for Plasticized Bituminous Hot-Mix Seal is included in this report as Appendix C. The field experimentation that led to permissive use of the plasticized seal in Illinois and subsequent evaluations of service performance is discussed in the second section of this Chapter.

PORTLAND CEMENT CONCRETE PAVEMENT TEXTURING

Based on the reported experiences of other agencies and confirmation from an experimental installation in Illinois, Illinois has replaced its formerly used burlap-drag method of applying a final finish to portland cement concrete pavement surfaces for frictional improvement with the much more positive artificial turf-transverse tining combination for

texturing. The new texturing procedure, after being in use for several years, was added to the Illinois Standard Specifications adopted October 1, 1979.

Under this specification, the final finish is obtained through the use of an artificial turf carpet dragged longitudinally over the surface of the plastic concrete and followed immediately by a mechanically operated metal comb transverse tining device. The tines are spaced at 1/2-in. centers and cut grooves 1/8 in. to 1/16 in. deep and 0.100 in. to 0.125 in. wide.

A summary description of the Illinois experimentation with pavement texturing and of subsequent evaluation of service performance is included in the second part of this chapter.

SURFACE GROOVING

Illinois has made occasional use of the grooving process for improving the friction quality of pavement surfaces at high wet-weather accident locations as a maintenance measure beginning at about the time the process came into use 15 or so years ago. It was applied at first to both portland cement concrete and bituminous concrete surfaces, but later confined to portland cement concrete surfaces after excessive wear under traffic was noted on many of the bituminous concrete surfaces. Results of evaluations of the effectiveness of the treatment in improving friction quality at a few of the treated locations have tended to be inconclusive.

EXPERIMENTATION - EVALUATION

Soon after beginning its systematic program of skid-accident reduction in 1964, the Illinois Department of Transportation began a varied program of field experimentation, and a limited amount of laboratory experimentation,

all of which has been directed toward the upgrading of the frictional characteristics of Illinois pavements through improved pavement design, construction, and maintenance procedures. Because of the strong influence that coarse-aggregate characteristics have on the friction quality of bituminous mixtures containing them, a major part of the experimentation has been concerned with the selection and use of suitable coarse aggregates in bituminous mixtures. The experimentation, in addition, has included studies of new pavement types, surface texturing of portland cement concrete pavements, and other items as well.

As new features have been introduced into practice, evaluations of service performance have been made to provide assurance that the innovations are serving as intended or to suggest necessary revisions.

Summary details of the individual experiments and service performance evaluations are presented in this section of Chapter 3. Most of the information presented is given in (9), (10), (11), and (14). A limited amount of the information was drawn from files.

CLASS I DENSE-GRADED BITUMINOUS CONCRETE SURFACE COURSE

A considerable amount of the experimentation and performance evaluation work that Illinois has done to improve friction quality has been concerned with Class I bituminous concrete surface course mixtures, and much of the work with these mixtures has been focused on the coarse-aggregate component. Experience and results of the sampling survey of 1969-71 had shown the greatest need for improvement to lie in this area.

Crushed Slag Coarse Aggregate Experiment - Troy

Crushed slag, as defined in the Illinois standard specifications, is the graded product resulting from the processing of air-cooled blast-furnace

slag. Air-cooled slag is a hard, vesicular material with a Moh's scale value in the 6-7 range. Some loss from abrasion can be expected under heavy traffic.

Slag coarse aggregate was included in four of eight experimental sections of bituminous concrete installed on a resurfacing project on Route US 40 east of Troy in Madison County in 1971. Route US 40 at the location is a two-lane pavement with an ADT volume of about 2,200. Of the eight test surfaces, three met Class I surface mixture requirements, three met Class I binder mixture requirements, and two were sand-asphalt mixtures (modified Class I). Suggested mixing formulas are given in Table 6; typical hot-bin analyses are given in Table 7.

Referring to Table 6, it will be seen that Mix 1 closely resembles today's Class C mix using limestone coarse aggregate, Mix 2 closely resembles the Class E mix with slag aggregate, and Mix 3 resembles the Class D mix using the slag-limestone coarse-aggregate combination.

When planning the study, it was believed that varying the amount and size of coarse aggregate retained on the No. 10 sieve in dense-graded mixtures would have a perceptible influence on macrotexture. Measurements of macrotexture of the finished surfaces by the putty impression method showed only minor increases in macrotexture with increase in aggregate size within the limits of the study.

Of the two surface course mixtures that contained slag, one contained an all-slag coarse aggregate that actually ranged from 3/8 in. to dust. The other contained about a 1:1 mixture by weight of the same slag aggregate

TABLE 6

SUGGESTED MIXING FORMULAS FOR CLASS I BITUMINOUS CONCRETE MIXTURES - TROY (9)

Type Aggregate and Gradation	Nominal Average Percent of Total by Weight									
	Surface Courses			Sand Courses			Binder Courses			
	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8		
Coarse Aggregate CA-7 (1-in. max.)	-	-	-	-	-	30(LS)	30.4(S)	41(S)		
CA-16(3/8-in. max.)	63(LS)	-	42(LS)	-	-	15(LS)	13.6(LS)	-		
3/8-in. to dust	-	70(S)	37.5(S)	-	-	-	-	40(S)		
Coarse Sand FA-1 (No. 4 max.)	19	9	-	82.6(LS)	-	17	16.3	-		
3/16-in. to dust	-	-	-	-	73(S)	-	-	-		
Fine Sand FA-10 (No. 40 max.)	10	9	14	9	14	29.3	29.4	15		
Mineral Filler	3.2	4.5	-	-	4.6	4.2	5.5	-		
Asphalt Content	4.8	7.5	6.5	8.4	8.4	4.5	5.8	7.2		

(LS) Limestone Aggregate
(S) Air-cooled blast furnace Slag

Mix Identification

- | | | |
|-------------------------|---------------------------|-------------------------|
| 1. Limestone | 4. Stone Sand (Limestone) | 6. Limestone |
| 2. Slag | 5. Slag | 7. Limestone-Slag Blend |
| 3. Limestone-Slag Blend | | 8. Slag |

TABLE 7

TYPICAL HOT-BIN ANALYSIS FOR CLASS I BITUMINOUS CONCRETE MIXTURES - TROY (9)

Sieve Size		Percent Retained by Weight											
		Surface Courses			Sand Courses			Binder Courses					
		Mix	Mix	Mix	Mix	Mix	Mix	Mix	Mix	Mix	Mix	Mix	Mix
Passing	Retained	1	2	3	4	5	6	7	8				
1 in.	3/4 in.	-)	-)	-)	-)	-)	8.2)	8.2)	6.6)				
3/4 in.	1/2 in.	-)	-)	-)	-)	-)	24.1)	28.4)	24.8)				
1/2 in.	No. 4	37.1)	23.0)	35.3)	0.2)	7.8)	24.3)	10.9)	22.6)				
No. 4	No. 10	24.3)	23.5)	30.2)	3.0)	32.1)	5.7)	14.6)	13.0)				
No. 10	No. 40	20.3)	24.4)	8.7)	45.2)	22.0)	17.8)	16.0)	14.7)				
No. 40	No. 80	7.0)	9.7)	7.9)	23.2)	8.7)	7.9)	6.4)	5.3)				
No. 80	No. 200	2.6)	5.5)	9.4)	16.7)	12.5)	2.9)	4.6)	4.6)				
No. 200		3.9	6.4	2.0	3.3	8.1	4.6	5.1	1.4				
Asphalt Content (AC 70-85)		4.8	7.5	6.5	8.4*	8.8	4.5	5.8	7.0				

* Bitumen (AC 85-100)

Mix Identification

- | | | |
|-------------------------|---------------|-------------------------|
| 1. Limestone | 4. Stone Sand | 6. Limestone |
| 2. Slag | 5. Slag | 7. Limestone-Slag Blend |
| 3. Limestone-Slag Blend | | 8. Slag |

and limestone coarse aggregate (Moh's scale hardness in the 3-4 range). Of two binder mixtures serving as surfaces and containing slag, one used an all-slag coarse aggregate (Mix 8), and the other about a 1:2 by weight limestone-slag coarse-aggregate combination (Mix 7). The third binder mix included as a surfacing in the study contained all-limestone coarse aggregate (Mix 6). Completing the study were the two sand-asphalt mixes, one with an all-limestone coarse sand fraction (Mix 4), and the other with an all-slag coarse sand fraction that actually graded from 3/16 in. to dust (Mix 5).

In this study, friction tests were made yearly from construction in 1971 through 1977 at the standard speed of 40 mph (FN_{40}), and also at 30 mph (FN_{30}), and 50 mph (FN_{50}) to provide friction number-speed gradients believed by some investigators to be useful in assessing the influence of macrotexture on friction quality. Friction number-speed gradients (G) were determined as suggested in FHWA Instructional Memorandum 21-2-73 where

$$G_{A-B} = \frac{FN_A - FN_B}{B - A}$$

with A and B being the test speeds at which the friction numbers are determined. In the present instance, the equation becomes

$$G = \frac{FN_{30} - FN_{50}}{50 - 30}$$

The results of the standard FN_{40} tests are given in Table 8. Friction number-speed gradients derived from the FN_{30} and FN_{50} are given in Table 9.

An examination of the results recorded in Tables 8 and 9 produces several items of interest as follows:

TABLE 8

SUMMARY OF FRICTION TEST RESULTS

Slag Experiment on Route US 40 near Troy (9)

Mix Number	July 1971			Nov. 1971			June 1972			Aug. 1973			July 1974			July 1975			June 1976			June 1977		
	30	40	50	30	40	50	30	40	50	30	40	50	30	40	50	30	40	50	30	40	50	30	40	50
1	45	40	38	45	42	38	51	45	38	41	37	33	41	36	32	41	38	36	39	36	33	46	39	33
2	38	35	32	42	38	34	49	42	37	51	40	36	50	43	38	56	48	44	52	42	36	67	59	49
3	53	51	48	54	51	46	57	51	44	50	43	39	44	39	36	49	44	40	45	40	34	56	49	42
4	39	36	31	40	33	28	38	31	24	30	23	20	34	27	21	30	32	23	36	27	21	50	36	26
5	51	52	43	54	48	42	52	44	38	48	41	35	44	38	33	53	50	40	52	43	36	70	59	47
6	31	29	26	36	30	27	42	34	30	36	31	26	35	30	26	35	31	28	35	30	26	43	36	28
7	42	39	37	46	39	35	51	45	37	48	41	37	44	40	35	57	46	44	51	42	35	64	57	50
8	47	44	41	52	45	44	56	50	44	45	38	34	48	39	35	57	45	43	52	42	35	65	57	48
2A*	52	46	43	-	-	-	60	51	44	58	49	42	54	45	37	-	-	-	56	48	36	70	63	54

*Mix Design Number 2 placed in eastbound lane at east end of project as 2A.

<u>Surface courses</u>		<u>Sand mixes</u>		<u>Binder courses</u>	
1.	Limestone	4.	Stone sand	6.	Limestone
2.	Slag	5.	Slag	7.	Limestone-slag
3.	Limestone-slag			8.	Slag

TABLE 9
FRICTION NUMBER-SPEED GRADIENTS
Slag Experiment on Route US 40 Near Troy

<u>Test Site and Mix Number</u>	<u>Bituminous Mixture</u>	<u>Coarse Aggregate</u>	<u>Friction Number- Speed Gradient</u>	
			<u>July 1971</u>	<u>June 1977</u>
1	Class I Surface	Limestone	0.35	0.65
2	Same	Slag	0.30	0.90
3	Same	Limestone-Slag	0.25	0.70
4	Class I Modified ^{2/}	Limestone ^{3/}	0.40	1.20
5	Same	Slag ^{3/}	0.40	1.18
6	Class I Binder	Limestone	0.25	0.75
7	Same	Limestone-Slag	0.25	0.70
8	Same	Slag	0.30	0.85
2A ^{1/}	Class I Surface	Slag	0.30	0.80

1/ Mix No. 2A at different site, same project (added to study when excessive flushing was noted at Test Site 2)

2/ Class I modified to sand mixture

3/ Sand size

- (1) Looking at the results of the standard skid tests made at 40 mph (Table 8), all test sections containing only limestone aggregates at some time dropped below the "satisfactory" level of friction quality as used in all Illinois studies. With the exception of a single reading on one section, the readings on all sections have remained within the satisfactory range through six years of service. Friction numbers at the 40 mph measuring speed have been significantly and consistently higher on the sections containing slag aggregate throughout the study, with neither the all-slag aggregate nor the limestone-slag combinations consistently showing the better performance of the two.
- (2) Again looking at the results of the standard 40 mph skid tests, it will be seen that none of the test sections showed a degradation of friction quality in six years of service and most showed improvement. This differs from common experience elsewhere in Illinois, perhaps as the result of the favorable influence of weathering predominating over the adverse influence of traffic wear because of the relatively low volume of traffic at the location.
- (3) Referring to Table 9, it will be noted that the computed friction number-speed gradient values have increased markedly for all test sections, and particularly for the two sand-asphalt sections, in six years of traffic service. Looking at the 1977 data in Table 8 and Table 9, it would appear that none of the mixes at that time could be depended upon to provide a high degree of skid resistance at high speeds, with the limestone aggregate mixes appearing to have the least capability for doing so.

- (4) The difference in the maximum aggregate size of the surface and binder mixtures does not appear to have had a substantial influence on the FN values.
- (5) With respect to the effect of the amount of material retained on the No. 10 sieve on friction quality, not much is apparent from the study except that the all-limestone coarse sand mix with very little material retained on the No. 10 sieve proved to have the lowest FN values of all the mixtures tested.

Class I, Mixture D and Mixture E Evaluations

Specification Mixtures C, D, and E and the policies under which they are used are outlined in the first section of this chapter.

The frictional characteristics of Mixture C, and of Mixture D when crushed dolomite and crushed gravel coarse aggregates are used, have been fairly well established because of their representation in the Statewide sampling survey. Less is known about the newer Mixture D containing equal volumes of crushed limestone and slag coarse aggregates, and Mixture E which may contain either fully crushed slag coarse aggregate, or crushed slag coarse aggregate blended equally by volume with other specified aggregates except crushed limestone.

Not many contractors constructing Mixture D surfacings have chosen to use the elective blend of equal parts by volume of crushed slag and crushed limestone coarse aggregates, with the result that friction-test data are available at the present time for only two such locations. One of these is a spot improvement on Route US 51 north of Sandoval and the other a spot improvement on Route Illinois 1 north of Carmi. FN_{40} values for the Route US 51 project ranged between 40 and 51 and averaged 46 shortly after

construction, and for the Route Illinois 1 project between 29 and 42, with an average of 37. It is of interest that the Route Illinois 1 surfacing that was tested replaced a similar surfacing placed a short time earlier that failed because the slag aggregate had not been dried sufficiently before use. The only other slag-limestone coarse aggregate blend tested is that near Troy reported in the immediately preceding section of this report.

Very limited use also has been made of the crushed slag-crushed dolomite coarse aggregate combination that may be used where Mixture E is specified. Friction test data for this alternative are available only for resurfacings placed in 1974 (before the formal adoption of Mixture E) on the Dan Ryan, Calumet, and I-57 expressways in the Chicago area. Tests on the Dan Ryan were made between 31st and 35th Streets; on the Calumet between 95th and 111th Streets, and on I-57 between 95th and Halsted Streets. Results of tests made in 1974 immediately following construction, and also in 1975, plus additional testing on the Dan Ryan in 1977 and 1979, are given in Table 10. It will be seen in the table that the recorded FN_{40} values were quite high immediately after construction, but dropped appreciably by the next year. They remained, however, with one exception, within the marginal range, even on the Dan Ryan where measurements were made five years after construction. Traffic on these expressways is of an intensity that exposes the surfaces to amounts of wear and polishing rarely experienced elsewhere.

A computation of mean wet-pavement/dry-pavement accident ratios for three years preceding and for three years following resurfacing of the Dan Ryan with the dolomite-slag coarse aggregate blend Class I mixture and of ratios for the same periods for the Kennedy and Eisenhower Expressways that received no treatments during the period produced the following results:

TABLE 10

SUMMARY MEASURED FRICTION QUALITY OF SLAG-DOLOMITE SURFACES
ON DAN RYAN, CALUMET, AND I-57 EXPRESSWAYS

(FN₄₀)

<u>Test Date</u>	<u>Southbound</u>	<u>Northbound</u>
	<u>DAN RYAN</u>	
10-23-74	59	--
6-25-75	41	41
11-19-75	36	35
7- 8-77	31	30
9-28-79	33	31
	<u>CALUMET</u>	
10-23-74	64	--
6-25-75	30	38
8-19-75	30	32
11-19-75	29	31
	<u>I-57</u>	
10-23-74	64	--
6-25-75	48	42
8-19-75	33	32

<u>Years</u>	<u>Mean Wet-Pavement/Dry-Pavement Accident Ratio</u>		
	<u>Dan Ryan</u>	<u>Kennedy</u>	<u>Eisenhower</u>
1970-72	0.26 ^{1/}	0.27	0.26
1974-76	0.19 ^{2/}	0.27	0.25

1/ Before resurfacing

2/ After resurfacing

It will be seen from the above that all three expressways showed about the same wet-pavement/dry-pavement accident ratios in the period before the Dan Ryan was resurfaced, and that only the ratio for the Dan Ryan showed a substantial drop in the second set of readings. The improvement seems logically attributable to the application of the bituminous concrete surfacing mixture containing the blend of equal proportions by volume of crushed slag and crushed dolomite coarse aggregates.

Although additional evaluative studies appear desirable, it seems at this time that the Class I Mixture E blend of equal parts by volume of crushed slag and crushed dolomite coarse aggregates probably is capable of providing generally satisfactory friction under the intended conditions of use.

Data regarding the frictional characteristics of the Mixture E all-slag coarse aggregate alternative are also of limited quantity. From the time the specification was adopted in May 1975 until the close of 1977, this alternative had been used on only 15 completed construction projects, all in District 1. The coarse aggregate in all instances was required to be the larger of the two sizes permitted (1/2-in. nominal maximum rather than 3/8-in. nominal maximum). All of the projects are relatively

small, with several being intersection improvements. All are located on low-speed, dense-traffic roadways presenting difficult friction-testing conditions, and only six have been tested to date. Summary results of the testing are shown in Table 11. It will be seen from the table that the measured FN_{40} values are nearly all at the established satisfactory level or near the top of the marginal level. Although none of the surfaces have been in service for more than a few years, previous experience with slag coarse aggregates in Illinois suggests that the friction numbers recorded after a few years of service may remain about the same over a considerable period of time.

It is of interest that comparative friction tests made on an adjoining Class I surfacing on I-55 with all-dolomite coarse aggregate constructed a year earlier than the all-slag coarse aggregate section for which FN_{40} values are shown in Table 11 and serving only slightly higher traffic volumes showed FN_{40} values 12 to 14 numbers lower.

Although additional evaluation is desirable, it appears at this time that the all-slag coarse aggregate alternative of Mixture E, like the alternative of using equal portions by volume of slag and dolomite coarse aggregates, probably is capable of providing generally satisfactory friction quality under the intended conditions of use.

Costs.--In February 1980, the Bureau of Design had occasion to prepare cost estimates for several currently used and potential bituminous overlay systems including different coarse aggregates in the surface courses. Results of the work are shown in Table 12. The estimate was prepared primarily to examine the additional cost of using crushed traprock aggregate in surface courses. Traprock is a hard igneous material derived from

TABLE 11

SUMMARY RESULTS OF FRICTION TESTS OF CLASS I
SURFACE COURSE MIXTURE E CONTAINING
ALL-SLAG COARSE AGGREGATE

<u>Location</u> <u>(District 1)</u>	<u>Date</u> <u>Completed</u>	<u>ADT</u>	<u>Tested</u>	Average FN_{40} <u>in Traffic Lane</u>
Dempster (Crawford to Skokie)	5-77	30,000	7-77 7-78 9-79 8-80	40 30 34 34
I-55 nb (0.4 m s. to 0.7 m n. I-80)	9-77	29,400	7-78 9-79 8-80	47 42 40
US 20 (Itasca to Mill Rd.)	9-76	20,000	7-78 9-79 8-80	40 34 39
Western Ave. (87th to 99th Sts.)	8-77	23,000	7-78 9-79 8-80	38 37 36
Ill. 83 (bridge deck over DesPlaines River)	8-78	22,200	9-79 8-80	42 35
Ill. 19 (Tollway to Cumberland)	11-77	30,000	9-79 8-80	36 34
Lake Shore Drive (Roosevelt Rd. to 53rd St.)	11-79	102,500- 53,600	6-80	47
Lake Shore Drive (53rd St. to 71st St.)	11-79	53,600- 23,000	6-80	46

TABLE 12

ESTIMATED COST PER MILE FOR BITUMINOUS OVERLAY SYSTEMS WITH
SURFACE COURSE MIXTURES OF DIFFERENT COARSE AGGREGATES

(February 1980)

<u>Mixture C</u>	<u>2-Lane</u>	<u>4-Lane</u>
Surface 1 1/2" Class C	\$ 35,182	\$ 70,364
Binder 1 1/2"	<u>34,165</u>	<u>68,330</u>
	\$ 69,347	\$138,694
 <u>Mixture D</u>		
Surface 1 1/2" Class D	\$ 36,519	\$ 73,038
Binder 1 1/2"	<u>34,165</u>	<u>68,330</u>
	\$ 70,684	\$141,368
 <u>Mixture E</u>		
100% Slag Surface 1 1/2"	\$ 39,179	\$ 78,358
Binder 1 1/2"	<u>34,165</u>	<u>68,330</u>
	\$ 73,344	\$146,688
 <u>Dense-Graded Traprock</u>		
100% Traprock Surface 1 1/2"	\$ 69,838	\$139,676
Binder 1 1/2"	<u>34,165</u>	<u>68,330</u>
	\$104,003	\$208,006
 <u>Open-Graded Traprock*</u>		
100% Traprock 5/8"	\$ 32,120	\$ 64,240
Binder 3"	<u>68,330</u>	<u>136,660</u>
	\$100,450	\$200,900

*Nonstructural open-graded asphalt friction course

ore separation and available to Illinois from one source in Missouri and one source in Wisconsin. It has been used in Illinois in open-graded asphalt friction courses as discussed in a following section of this report, but not in Class I construction. As will be seen from the table, it is an expensive material. Of further interest in the table is the indication that the introduction of Mixtures D and E narrowing the choice of coarse aggregates for upgrading skid resistance has not added greatly to construction costs.

Synopal Coarse Aggregate

During the years 1965-69, a number of trial installations of Class I bituminous concrete surfaces using a coarse aggregate identified as "Synopal" were placed under resurfacing contracts. Synopal is a very white and very hard vesicular aggregate manufactured only in Denmark at the time under a proprietary arrangement. It is produced by melting sand, chalk or limestone, and dolomite, cooling the molten mass to a granular state, and then reheating to a temperature close to the melting point. Individual particles are irregularly cubic in shape, have a crushing strength 40 percent above that of granite, and a Moh's scale hardness of 7.5 (quartz is 7).

Although costing several times more than local aggregates, the potential that Synopal appeared to have for greatly improving both night visibility and frictional quality, and a possibility that a manufacturing plant would be constructed in the Illinois vicinity (which did not materialize) to reduce its cost, induced the experimentation.

Test sections in which Synopal aggregate was substituted for parts of the limestone coarse aggregate in Class I bituminous concrete surfacing mixtures were established in 1965 under resurfacing contracts on the then heavily

traveled Route US 66 south of Pontiac, and in 1969 on the collector-distributor lanes of the Dan Ryan Expressway 6 miles south of Chicago's Loop. On Route US 66, sections in which the Synopal replaced 25 and 50 percent by volume of the limestone coarse aggregate were established. (Wide differences in specific gravities made volume rather than weight replacement desirable.) On the Dan Ryan Expressway, a test section having a 50 percent by volume replacement of the crushed stone coarse aggregate with Synopal was established.

Friction tests in the traffic lanes at Pontiac made in 1969 showed the 50 percent Synopal section to be in the satisfactory range and both the 25 percent Synopal section and a control section without Synopal to be in the low range. Later improvement placed the 25 percent Synopal section, but not the control section, in the marginal range. After six years of service and over 26.5 million axle applications, the 50 percent Synopal section continued to be in the satisfactory range 10 to 12 friction numbers above the control section in the low range and several numbers above the 25 percent Synopal section in the marginal range. The 50 percent Synopal section on the Dan Ryan Expressway showed friction quality well above the minimum satisfactory level in the beginning and near the top of the marginal level after 28 million axle applications.

The results of the Synopal aggregate experiments were concluded to show that the substitution of Synopal in the amount of 50 percent by volume of limestone coarse aggregates in common use in Illinois can provide significant improvements in friction quality. Subjective observations also showed Synopal aggregate to have a very favorable influence on night visibility.

A very high cost that apparently was to continue discouraged further use of the material.

OPEN-GRADED ASPHALT FRICTION COURSE

The open-graded asphalt friction course (OGAFC) is a derivation of what originally was conceived to be a plant-mix seal coat. Under the earlier concept, the new layer was expected to provide a waterproofing seal for the existing surface in addition to serving the other functions of a good riding surface including provisions of adequate structural support, riding quality, and skid resistance. The waterproofing function was achieved by incorporating enough asphalt in the mixture to allow flow to seal the older surface. Under the new concept, sealing of the existing surface is evaluated and treated separately, and the new surface is designed to best serve its other functions. The new system eliminates some of the uncertainties inherent in the original-function concept.

Because of its open texture, the OGAFC is considered to have a special additional capability for eliminating the danger of hydroplaning in heavy rainstorms by providing channels for easy water escape. At the time that the OGAFC was first used in Illinois in 1974, little was known by Illinois engineers and contractors about the behavior of OGAFC mixtures during the construction process, and service performance using materials available to Illinois and serving under local conditions could only be speculated upon. As a consequence, all of the installations of OGAFC to date (there was a total of 71 sections constructed at 52 sites through 1978) can be considered to be in the nature of trial installations.

In the Illinois work, specifications for material selection, the procedure for designing the OGAFC mixtures, and construction procedures that have been used, have been in general accord with FHWA and Asphalt

Institute recommendations. As information has been gained, certain coarse aggregates have been eliminated from further use, application directly on portland cement concrete surfaces has been discontinued, and use under poor drainage conditions is now avoided, all following observations of deficient performance. Limitations also have been placed on use under certain traffic conditions.

Among the principal construction variables included in the installations to date has been the type of coarse aggregate used. The following types have been included:

<u>Aggregate Type</u>	<u>Description</u>
(1) Crushed steel slag	Graded product resulting from the processing of open-hearth or basic-oxygen furnace slag
(2) Crushed traprock	Graded product resulting from processing of a dark gray igneous material resulting from ore separation
(3) Crushed slag	Graded product resulting from the processing of air-cooled blast furnace slag
(4) Novaculite gravel	Material from natural deposits composed of angular particles of siliceous origin mixed with ferruginous clay
(5) Materialite	A manufactured aggregate derived from calcined shale
(6) Crushed gravel	Product resulting from the crushing of gravel by mechanical means
(7) Crushed dolomite	Product resulting from the crushing of dolomite by mechanical means

The first use of OGAFc in Illinois in 1974 included placement at four high-accident sites. In 1975, two experimental sites of OGAFc were constructed in addition to four other sites at high-accident locations. One of the experimental sites, located on the South Shore Drive in Chicago, includes a test section in which crushed traprock was used as the coarse

aggregate and a test section having Materialite as the coarse aggregate. The other experimental site, located on Route Illinois 143 between Wood River and Edwardsville contains four test sections with different coarse aggregates as follows: crushed steel slag, crushed slag, crushed trap-rock, and Materialite. In 1976, two more experimental sites incorporating crushed slag, crushed gravel, and crushed dolomite coarse aggregates were constructed on I-55 in the northbound lanes south of Route Illinois 53 and in both the northbound and southbound lanes at California Avenue. The remainder of the 52 sites constructed through 1978 were at either high-accident locations or bridges. Most sites are relatively short segments less than a half mile in length. ADT volumes range from about 1,600 to 130,000. All site locations are identified in Table 13 and shown on the map of Figure 17.

A survey of the general condition of all OGAFS sites constructed through 1978, varying in age from one to five years, was conducted in May 1979. The types of distress and deficiencies observed during the survey are indicated by the headings in Table 14 where the extent and severity of each is categorized for all sections individually. Observed structural losses included raveling (area raveling, popout raveling, and raveling at joints, cracks and pavement edges) and delamination. Observed deficiencies affecting permeability included loss of void space (openness), flushing, and fat spots. Observed overall performance as related to wheel-path and total pavement area also is indicated in Table 14. All of the terms used to characterize the observed types of distress and deficiencies, with the exception of the term "popout raveling," have been used by other

TABLE 13

IDENTIFICATION AND LOCATION OF OGAFS SITES (11)

SITE	ROUTE	COUNTY	LOCATION	AGGREGATE	ADT	CONST.	
						YEAR	% AC
1-01-MA-5	US 41	Cook	US 41 (South Shore Drive) from Cheltenham Place to 80th St.	Materialite	7,700	1975	9.10
1-01-TR-5	US 41	Cook	US 41 (South Shore Drive) from 78th St. to Cheltenham Place.	Traprock	7,700	1975	7.0
1-02-SA-5	FAI 55	Will	Northbound I-55 to Eastbound I-80 Ramp.	Air-cooled Slag	2,500	1975	9.0
1-03-TR-5	FAI 80	Will	FAI 80 West of Des Plains River.	Traprock	22,000	1975	7.0
1-04-SA-6	FAI 55	Cook	FAI 55 at California Ave.	Air-cooled Slag	130,000	1976	9.0
1-04-CG-6	FAI 55	Cook	FAI 55 at California Ave.	Crushed Gravel	130,000	1976	6.5
1-04-DM-6	FAI 55	Cook	FAI 55 at California Ave.	Dolomite	130,000	1976	6.4
1-05-SA-6	FAI 55	Will	FAI 55 Northbound Lanes South of I11 53.	Air-cooled Slag	28,900	1976	9.0
1-05-CG-6	FAI 55	Will	FAI 55 Northbound Lanes South of I11 53.	Crushed Gravel	28,900	1976	6.5
1-05-DM-6	FAI 55	Will	FAI 55 Northbound Lanes South of I11 53.	Dolomite	28,900	1976	6.4
1-06-CG-6	I11 53	Will	I11 53 at Naperville Rd. (Overlaid)	Crushed Gravel	14,100	1976	6.5
1-07-MA-6	I11 83	Dupage	I11 83 at St. Charles (Overlaid)	Materialite	31,200	1976	9.0
1-08-TR-6	US 41	Lake	US 41 at Clavey Rd.	Traprock	41,400	1976	6.0
1-09-TR-6	US 41	Lake	US 41 at West Park Ave.	Traprock	41,400	1976	6.0
1-10-TR-6	US 41	Lake	US 41 at I11 22	Traprock	41,400	1976	6.0
1-11-TR-6	US 41	Lake	US 41 at I11 137	Traprock	27,000	1976	6.0
1-12-TR-6	US 12	Lake	US 12 at I11 22	Traprock	22,500	1976	6.0

TABLE 13

IDENTIFICATION AND LOCATION OF OGAFS SITES (cont'd)

SITE	ROUTE	COUNTY	LOCATION	AGGREGATE	ADT	CONST.	
						YEAR	% AC
2-01-TR-4	US 51	Winnebago	US 51 at Ill 173 North of Rockford	Traprock	15,000	1974	7.0
2-02-TR-6	US 20	Stephenson	US 20 at Turner Ave. in Freeport	Traprock	19,500	1976	6.3
2-03-TR-6	Ill 75	Stephenson	Ill 75 North of Freeport	Traprock	4,500	1976	6.3
2-04-TR-7	US 51	Winnebago	US 51 at Blackhawk Road South of Rockford	Traprock	12,000	1977	6.3
2-05-SA-8	Ill 2	Lee	Ill 2 at River Road in Dixon	Air-cooled Slag	9,500	1978	9.5
3-01-MA-6	FAI 57	Iroquois	FAI 57 Bridge Decks over Twp Rd North of US 24	Materialite	11,500	1976	10.0
3-02-MA-6	FAI 57	Iroquois	FAI 57 Bridge Decks over TP&W RR North of US 24	Materialite	11,500	1976	10.0
3-03-MA-6	FAI 57	Kankakee	FAI 57 Bridge Decks over Ill 50	Materialite	14,600	1976	10.0
3-04-MA-6	FAI 57	Kankakee	FAI 57 Bridge Decks over ICG RR North of Ill 50	Materialite	14,000	1976	10.0
3-05-MA-6	US 24	Iroquois	US 24 West of Watseka	Materialite	6,100	1976	10.0
3-06-MA-6	Ill 47	Grundy	Ill 47 at Mazon	Materialite	5,700	1976	9.6
3-07-DM-7	US 51	McLean	US 51 North of FAI 55, Bloomington	Dolomite	9,600	1977	7.0
3-08-DM-7	US 150	McLean	US 150 at Leroy	Dolomite	4,100	1977	7.0
3-09-DM-7	US 150	McLean	US 150 at Downs	Dolomite	2,400	1977	7.0
3-10-TR-7	FAI 55	McLean	FAI 55 Loop over FAI 74	Traprock	7,800	1977	6.3
3-11-DM-7	Ill 57	Kendall	Ill 47 at US 52	Dolomite	3,150	1977	6.3

TABLE 13

IDENTIFICATION AND LOCATION OF OGAFS SITES (Cont'd)

SITE	ROUTE	COUNTY	LOCATION	AGGREGATE	ADT	CONST. YEAR	% AC
4-01-SA-7	FAI 74	Knox	Curves on FAI 74 East of Knoxville	Air-cooled Slag	9,600	1977	9.1
4-02-SA-7	FAI 74	Knox	Curves on FAI 74 East of Galesburg	Air-cooled Slag	9,600	1977	9.1
5-01-TR-4	US 36	Macon	US 36 at Staley Bridge in Decatur	Traprock	9,600	1974	7.0
5-02-TR-6	ILL 121	Macon	ILL 121 at US 36 in Decatur	Traprock	16,900	1976	6.5
5-03-TR-7	US 36	Macon	US 36 at Mohroe in Decatur	Traprock	22,300	1977	6.1
5-04-TR-8	ILL 148	Macon	ILL 148 at Main Street in Decatur	Traprock	17,200	1978	6.1
6-01-TR-7	FAI 55 Loop	Sangamon	South 6th St. in Springfield	Traprock	33,200	1977	6.2
8-01-TR-5	ILL 143	Madison	ILL 143 Wood River to Edwardsville	Traprock	8,500	1975	6.5
8-01-MA-5	ILL 143	Madison	ILL 143 Wood River to Edwardsville	Materialite	8,500	1975	9.5
8-01-SA-5	ILL 143	Madison	ILL 143 Wood River to Edwardsville	Air-cooled Slag	8,500	1975	8.5
8-01-SS-5	ILL 143	Madison	ILL 143 Wood River to Edwardsville	Steel Slag	8,500	1975	7.5
9-01-TR-4	ILL 148	Williamson	ILL 148 at Herrin	Traprock	13,000	1974	6.5
9-02-TR-4	ILL 13	Jackson	Old ILL 13 in Carbondale (Overlaid)	Traprock	-	1974	6.5
9-03-TR-5	ILL 146	Alexander	ILL 146 at Cape Girardeau	Traprock	8,000	1975	6.5
9-04-TR-5	US 51	Pulaski	US 51 at Cairo (Temporary Detour Removed)	Traprock	-	1975	6.5
9-05-NV-6	ILL 127	Perry	ILL 127 at ILL 152	Novaculite	4,000	1976	8.3
9-06-TR-6	US 51	Union	US 51 South of ILL 146	Traprock	2,400	1976	6.5
9-07-NV-7	ILL 148	Williamson	ILL 148 North of Herrin	Novaculite	14,500	1977	8.3

TABLE 13

IDENTIFICATION AND LOCATION OF OGAFC SITES (cont'd)

SITE	ROUTE	COUNTY	LOCATION	AGGREGATE	ADT	CONST. YEAR	% AC
9-08-NV-7	US 51	Pulaski	US 51 at Mounds	Novaculite	1,600	1977	8.5
9-09-NV-7	US 51	Pulaski	US 51 North of Mounds	Novaculite	1,600	1977	8.5
9-10-NV-7	US 45	Saline	US 45 North of Harrisburg	Novaculite	8,300	1977	8.5
9-11-NV-7	Ill 154	Perry	Ill 154 East of Pinckneyville	Novaculite	6,100	1977	8.3
9-12-TR-7	Ill 149	Jackson	Ill 149 at Kinkaid Hill East of Ill 3	Traprock	2,500	1977	6.5
9-13-NV-7	US 51	Union	US 51 South of Ill 146	Novaculite	2,400	1977	8.5
9-14-NV-7	US 51	Union	US 51 at Dongola	Novaculite	2,400	1977	8.5
9-15-NV-8	Ill 13	Jackson	Old Ill 13 East of Murphysboro	Novaculite		1978	8.3
9-16-NV-8	US 45	Massac	US 45 North of Metropolis	Novaculite	7,000	1978	8.5

- ⊙ STEEL SLAG
- ⊕ TRAPROCK
- NOVACULITE
- AIR-COOLED SLAG
- * MATERIALITE
- CRUSHED GRAVEL
- DOLOMITE

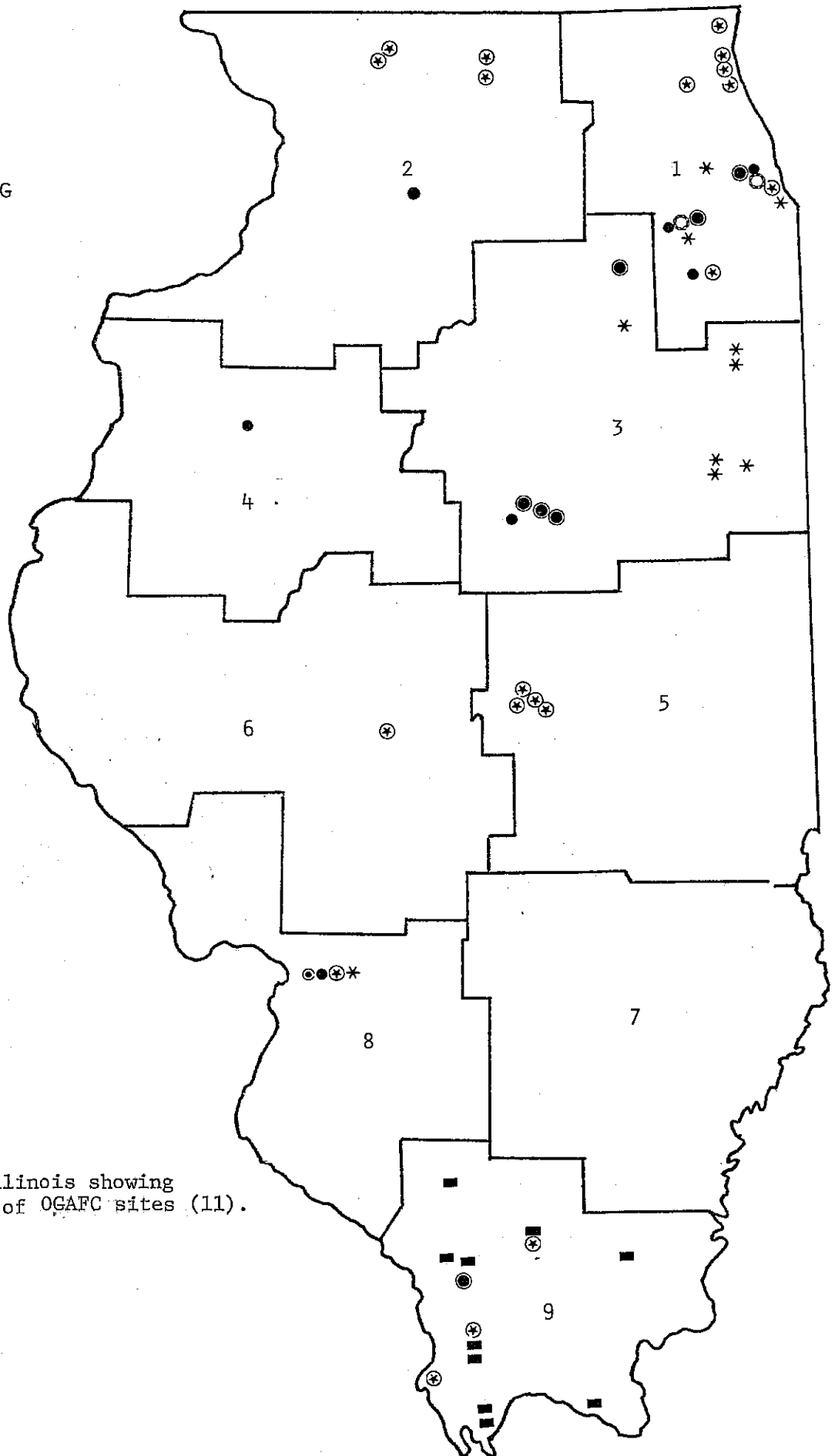


Figure 17. Map of Illinois showing location of OGAFS sites (11).

TABLE 14

SUMMARY OF CONDITION SURVEY OF OGAFC SECTIONS (11)

SECTION DESIGNATION	STRUCTURAL LOSS ^{1/}						PERMEABILITY			DISTRESS ^{1/}		EST. TOTAL VEHICLES (MILLION)	FN (INIT.)	FN (1978)	
	JOINTS	RAVELING CRACKS	EDGES	POPOUTS	AREA	DELAMINATION	SURFACE	FLUSHING	FAT SPOTS	WHEELPATH	TOTAL AREA % ASPHALT				
3-01M-SS-5	0	0	0	0	0	0	1	1	1	0	0	7.5	6.2	46	49
2-01M-TR-4	0	0	0	1	0	0	X	X	0	X	X	7.0	9.6	44	
5-01M-TR-4	X	X	X	0	X	X	1	1	0	X	X	7.0	13.9	37	
9-01M-TR-4	1	1	0	0	0	0	0	0	0	0	0	6.5	11.9	42	
9-02M-TR-4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
1-01M-TR-5	0	0	0	0	0	0	X	1	X	X	X	7.0	5.6	-	
1-03E-TR-5	0	0	1	0	0	1	X	1	0	1	0	7.0	17.5	52	38
1-03W-TR-5	0	0	1	0	0	X	X	X	0	X	X	7.0	17.5	52	38
8-01M-TR-5	0	0	0	0	1	0	0	1	1	0	0	6.5	6.2	48	38
9-03M-TR-5	0	0	0	0	1	0	1	1	0	0	0	6.5	5.8	40	
9-04M-TR-5	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
1-08S-TR-6	X	0	0	1	0	0	X	0	1	0	0	6.0	11.3	-	
1-08N-TR-6	1	0	0	0	0	0	X	0	0	0	0	6.0	11.3	-	
1-09S-TR-6	0	0	0	0	0	0	1	0	0	0	0	6.0	11.3	-	
1-09N-TR-6	1	0	0	0	0	0	1	0	0	0	0	6.0	11.3	-	
1-10S-TR-6	0	0	0	0	0	0	1	0	0	0	0	6.0	11.3	-	
1-10N-TR-6	0	0	0	0	0	0	0	0	0	0	0	6.0	11.3	-	
1-11N-TR-6	0	0	0	0	0	0	X	0	0	0	0	6.0	8.9	-	
1-12S-TR-6	0	0	0	0	0	0	0	0	0	0	0	6.0	7.4	-	
1-12N-TR-6	1	1	0	0	0	0	0	0	0	0	0	6.0	7.4	-	
2-02M-TR-6	0	0	0	1	0	0	0	1	1	1	1	6.3	7.5	-	
2-03M-TR-6	0	0	1	1	0	0	0	0	0	0	0	6.3	2.5	49	
5-02M-TR-6	1	1	1	0	1	0	0	0	X	0	1	6.5	6.1	-	
9-06M-TR-6	0	1	0	0	1	0	0	0	0	0	0	6.5	1.3	52	
2-04M-TR-7	X	0	X	1	0	0	0	0	1	0	1	6.3	3.1	-	
3-10M-TR-7	0	0	1	1	1	1	0	0	0	1	1	6.3	2.0	-	
5-03M-TR-7	0	1	0	0	1	1	1	1	0	0	0	6.1	5.7	-	
6-01M-TR-7	0	0	0	0	0	0	0	0	0	0	0	6.2	6.1	-	
9-12M-TR-7	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
5-04M-TR-8	0	0	0	0	0	0	X	0	0	0	0	6.1	3.0	-	
9-05M-NV-6	0	1	1	1	X	0	0	0	0	1	1	8.3	2.9	66	49
9-07M-NV-7	0	X	1	0	1	0	0	0	0	1	1	8.3	5.3	-	
9-08M-NV-7	0	X	1	0	1	0	0	0	0	1	1	8.5	0.6	-	
9-09M-NV-7	0	1	0	0	1	0	0	0	0	0	0	8.5	0.6	-	
9-10M-NV-7	0	0	0	1	1	0	0	0	1	0	0	8.5	3.0	-	
9-11M-NV-7	0	X	1	1	1	0	0	0	0	1	1	8.3	2.2	-	
9-13M-NV-7	1	1	0	0	1	0	0	0	0	0	0	8.5	0.8	-	
9-14-NV-7	0	X	1	0	X	0	0	0	0	1	1	8.5	0.8	-	
9-15M-NV-8	0	0	0	0	1	0	0	0	0	0	0	8.3	-	-	
9-16M-NV-8	0	0	0	0	0	0	0	0	0	0	0	8.5	1.3	-	

TABLE 14

SUMMARY OF CONDITION SURVEY OF OGAFS SECTIONS (cont'd)

SECTION DESIGNATION	STRUCTURAL LOSS ^{1/}						PERMEABILITY			DISTRESS ^{1/}					
	JOINTS	RAVELING CRACKS	EDGES	POPOUTS	AREA	DELAMINATION	SURFACE	FLUSHING	FAT SPOTS	WHEELPATH	TOTAL AREA	% ASPHALT	EST. TOTAL VEHICLES (MILLION)	FN (INIT.)	FN (1978)
1-02M-SA-5	0	0	1	0	0	1	0	0	X	0	X	9.0	3.7	50	
8-01M-SA-5	0	0	0	1	0	0	1	1	0	0	0	8.5	6.6	47	46
1-04S-SA-6	1	0	1	1	1	0	1	0	0	1	1	9.0	21.3	50	44
1-04N-SA-6	1	0	1	X	X	1	1	0	0	X	X	9.0	24.9	49	32
1-05N-SA-6	1	0	X	0	1	0	0	0	0	0	0	9.0	7.9	46	48
4-01E-SA-7	0	0	0	0	0	0	0	1	0	0	0	9.1	2.5	54	-
4-01W-SA-7	0	0	0	0	0	0	0	1	0	0	0	9.1	2.5	54	-
4-02E-SA-7	0	0	0	0	0	0	0	0	0	0	0	9.1	2.5	54	-
4-02W-SA-7	0	0	0	0	0	0	0	0	0	0	0	9.1	2.5	54	-
2-05M-SA-8	0	0	0	0	0	0	0	1	0	0	0			-	-
1-01M-MA-5	-	-	-	-	X	-	-	-	-	X	X	9.1	5.6	60	-
8-01M-MA-5	0	0	1	0	X	0	1	1	0	X	X	9.5	6.2	45	58
1-07M-MA-6	-	-	-	-	X	-	-	-	-	X	X	9.0	-	-	-
3-01N-MA-6	1	0	0	1	0	1	0	0	0	0	0	10.0	4.4	60	-
3-01S-MA-6	0	0	0	1	0	1	1	0	0	1	1	10.0	4.4	60	-
3-02N-MA-6	X	0	0	1	1	1	1	0	0	X	X	10.0	4.4	61	-
3-02S-MA-6	0	0	0	1	0	1	1	X	1	X	X	10.0	4.4	60	-
3-03N-MA-6	1	0	0	1	X	1	1	0	0	X	X	10.0	5.6	-	-
3-03S-MA-6	1	0	0	0	X	1	1	0	0	X	X	10.0	5.6	-	-
3-04N-MA-6	0	1	0	0	X	1	1	0	1	X	X	10.0	5.6	-	-
3-04S-MA-6	0	1	0	0	X	1	1	0	0	X	X	10.0	5.6	-	-
3-05M-MA-6	0	0	0	1	0	1	0	0	0	0	0	10.0	3.5	60	-
3-06M-MA-6	0	1	X	1	1	X	1	0	0	X	X	10.0	3.1	-	-
1-04S-CG-6	1	0	0	1	X	0	X	0	0	X	X	6.5	24.9	40	29
1-04N-CG-6	1	0	X	1	1	0	X	0	0	X	X	6.5	21.3	37	34
1-05N-CG-6	1	0	X	1	1	1	1	0	0	0	1	6.5	7.9	36	32
1-06M-CG-6	-	-	-	-	-	-	-	-	-	X	X	6.5	-	-	-
1-04N-DM-6	1	0	0	X	1	0	X	0	0	X	X	6.4	24.9	41	32
1-04S-DM-6	1	0	0	1	X	0	X	0	0	X	X	6.4	21.3	41	29
1-05N-DM-6	X	0	X	X	1	1	1	0	0	X	X	6.4	7.9	35	32
3-07M-DM-7	1	1	1	1	1	0	0	1	1	X	1	7.0	3.5	-	-
3-08M-DM-7	0	1	X	1	X	X	1	X	1	X	X	7.0	1.5	-	-
3-09M-DM-7	1	1	X	1	X	X	1	1	1	X	X	7.0	0.9	-	-
3-11M-DM-7	0	1	1	1	X	1	X	0	0	X	X	6.3	1.1	-	-

TABLE 14

SUMMARY OF CONDITION SURVEY OF OGAFS SECTIONS (cont'd)

1/ Definitions of Distress and Deficiency Ratings:

- | | |
|---|---|
| 0 | No raveling or delaminations, open surface, no flushing or fat spots. |
| 1 | Minor raveling along joints, cracks, and edges; some random popout raveling; occasional area raveling and delaminating (less than 1 sq ft areas); some gouges; a partly closed surface; a few small fat spots; no more than 5 percent of the traffic lane wheelpath length raveled and flushed; no more than 5 percent of the total pavement area affected by raveling, delaminating, flushing, and fat spots. |
| x | Major raveling along joints, cracks, and pavement edges; frequent popout raveling; large-size or numerous small-size area raveling; large delaminated areas; a closed surface; extensive flushing in the traffic lane wheelpaths; many fat spots; more than 5 percent of the traffic lane wheelpath length raveled and flushed; more than 5 percent of the total pavement area affected by raveling, delaminating, flushing, and fat spots. |

investigators to indicate the same kinds of defects. The term "popout raveling" was used to indicate a random loss of coarse aggregate particles. Three categories were established for classifying the various distress and efficiency factors, and overall performance, based on the extent of occurrence, as shown by the footnote on the table.

An estimate of the numbers of vehicles passing over each of the surveyed sections, and available asphalt contents and friction numbers, also are tabulated in Table 14.

A summary grouping of the 71 sections surveyed into three categories dependent on the nature and extent of observed distress and deficiencies, and separated by coarse aggregate type, is given in Table 15. For the purpose of constructing this table, three categories of distress or deficiencies were established as shown by the footnote on the table.

It will be noted in Table 15 that slightly over half of the sections surveyed were observed to have one percent or more of the total pavement area experiencing some form of distress or deficiency (Major and Minor Distress categories), and of these sections, over two thirds had in excess of 5 percent of the total area experiencing distress or deficiencies (Major Distress category).

It will be seen also from Table 15 that the type of coarse aggregate used in the OGAFc mixtures had a definite influence on performance. Mixtures containing steel slag, traprock, and air-cooled slag coarse aggregates appeared to be giving significantly better service as compared with mixtures containing any of the other coarse aggregates used. Following is a general discussion of the observed performance of the various

TABLE 15

SUMMARY GROUPING OF SURVEYED OGAFS SECTIONS BY OVERALL
CONDITION AND COARSE AGGREGATE TYPE (adapted from (11))

<u>Coarse Aggregate</u>	<u>Number of Sections</u>	<u>Area Distress^{1/}</u>		
		<u>Minute or None</u>	<u>Minor</u>	<u>Major</u>
Steel Slag	1	1	-	-
Traprock	26	18	4	4
Slag (Air-cooled)	10	7	1	2
Novaculite	10	6	4	-
Materialite	13	2	1	10
Crushed Gravel	4	-	1	3
Dolomite	7	-	1	6
Total	71	34	12	25

1/ Definitions of Area Distress Categories:

- Minute or None - Good overall appearance, open surface texture, no more than occasional and random loss of coarse aggregate particles, less than 1 percent of total area showing raveling, delaminating, flushing, and fat spots.
- Minor - From 1 to 5 percent of total area showing raveling, delaminating, flushing, and fat spots.
- Major - Over 5 percent of total area showing raveling, delaminating, flushing, and fat spots.

sections surveyed grouped by the types of coarse aggregate incorporated in the mixtures, and also some discussion of the four experimental sites that have been monitored in somewhat greater detail than the other sites.

Steel Slag

Only one section of steel slag exists in the State; it is one of four experimental sections on Route Illinois 143 between Wood River and Edwardsville. The surface, which was placed in 1975, was seen to be providing satisfactory service with evidence of only minor wheelpath flushing and partly closed surface.

Although the performance of the steel slag section was observed to be satisfactory at the time of the survey, the availability of only one section for study was considered to provide insufficient evidence for making any probable-life estimates. One concern about steel slag is its tendency to expand. Steel slag should be aged in stockpiles before it is used in a confined situation and in an alkaline environment because it expands as the calcium and the magnesium oxides hydrate. This expansion can be stabilized by treating it with spent steel by-product pickling liquors. Whether the steel slag used in this OGAFS was aged was not determined, but more sections using steel slag should be constructed for further evaluation.

In 1978, pavement friction averaged 49, which is 3 friction numbers above the initial value of 46.

Traprock

At the time of the survey, traprock had been observed longer (up to 5 years) in more sections than any other aggregate. Of the 26 sections surveyed, 70 percent had minute or no distress, 15 percent had minor overall

distress, and 15 percent involved major overall distress. The major distress that was observed was attributed mainly to a loss of permeability. Early construction projects (1974-75) contained higher than necessary asphalt contents, which resulted in wheelpath flushing in traffic lanes and near stop intersections, indicative of a closing of the surface texture. In early mix designs the asphalt content was not adjusted for the specific gravity of the aggregate. Later, when an adjustment was made, evidence of wheelpath flushing had appeared in only two of the 14 sites constructed since 1975. The absence of flushing at high-volume signalized intersections in Lake County and in Springfield was considered especially encouraging.

As for structural loss, limited amounts of major raveling caused mostly by snowplow abrasion were observed along longitudinal joints, pavement edges, and small isolated areas.

Major delaminations were seen in two sections placed directly on PCC pavements. One section is located where drainage is poor in a vertical sag of the westbound lanes of I-80 in Joliet. After the survey of this section was conducted, the delaminated areas were patched with a sand mix, which in some patched areas has been seen to be blocking internal drainage. The other section involves the approaches to the Route US 36 Staley Bridge in Decatur. The delaminations in this section were associated mainly with joints where surface water collects. These experiences were taken to indicate that delaminations are more likely to occur when OGAFc's are placed on PCC pavements, even with a tack coat, than when they are placed on bituminous surfaces.

Projects constructed after 1975 generally were performing satisfactorily even when subjected to high traffic volumes. Experience thus far was considered to indicate traprock to be one of the most promising aggregates

that can be used in OGAF's. It was estimated that a useful life of 10 years may be attainable.

Traprock comes from two sources: one is Dresser, Wisconsin and the other is Iron Mountain, Missouri. On the basis of pavement friction wear curves, the friction number for Dresser traprock averages in the mid 40's after 10 million axle applications while that for Iron Mountain is about 40, approximately 5 friction numbers lower (Figure 17). The difference in friction between the two traprocks was believed associated with grain and crystal size, where larger sizes provide higher friction. Although adequate, the friction provided by traprock was less than that for slags.

Air-cooled Slag

Air-cooled slag, which has been observed since 1975, had performed quite well. Of the 10 sections surveyed, 7 had minute or no distress, 1 had minor distress, and 2 had major distress. The major distress observed in the northbound I-55 to eastbound I-80 ramp resulted from fat spots, which covered about one third of the ramp between wheelpaths. These fat spots occurred during construction because of a high mixing temperature. Asphalt drain-down to the bottom of the truck bed occurred during transit. When the mix was dumped into the paver hopper, the rich part of the mix, in the center, could not be augered to the outer edges of the paver screed. This created fat spots in the center of the ramp.

Major distress also was observed in the northbound outer traffic lane of I-55 (Stevenson Expressway) near California Avenue where the outer wheel-path contained major area raveling and frequent popout raveling. This section had been subjected to higher volumes of heavy trucks than any other OGAF section surveyed.

Although minor raveling and flushing had occurred in some of the other sections, air-cooled slag surfaces, on the whole, were seen to be performing quite well. This aggregate, like traprock, was considered to show promise of reaching a service life of up to 10 years.

According to pavement friction wear curves developed for air-cooled slag (Figure 18), the average friction number at 10 million axle applications was approximately 48, which is slightly higher than that for Dresser traprock.

Novaculite

Novaculite (Jordan gravel) surfaces, most of which had been in service 2 years or less, were showing signs of minor overall distress at an early age. Of the 10 sections surveyed, 6 had minute or no distress, 4 contained minor overall distress, and none had major overall distress. Nevertheless, half of the sections showed evidence of major area raveling and major crack raveling, and most sections showed evidence of minor raveling after no more than 2 years of service. Raveling, which was most evident in the wheelpaths, was attributed to vehicular traffic dislodging aggregates from the surface. Novaculite's coarse texture was deemed probably to result in part from individual aggregates being dislodged from the surface. A theory offered for this raveling is that the aggregates may not have been thoroughly dried prior to mixing. The residual moisture could weaken the bond between the asphaltic binder and the aggregate.

Because of the minor raveling that had occurred after only 1 to 2 years of service, raveling in novaculite was expected to continue and perhaps develop into major distress before the surfaces provide 5 years of service.

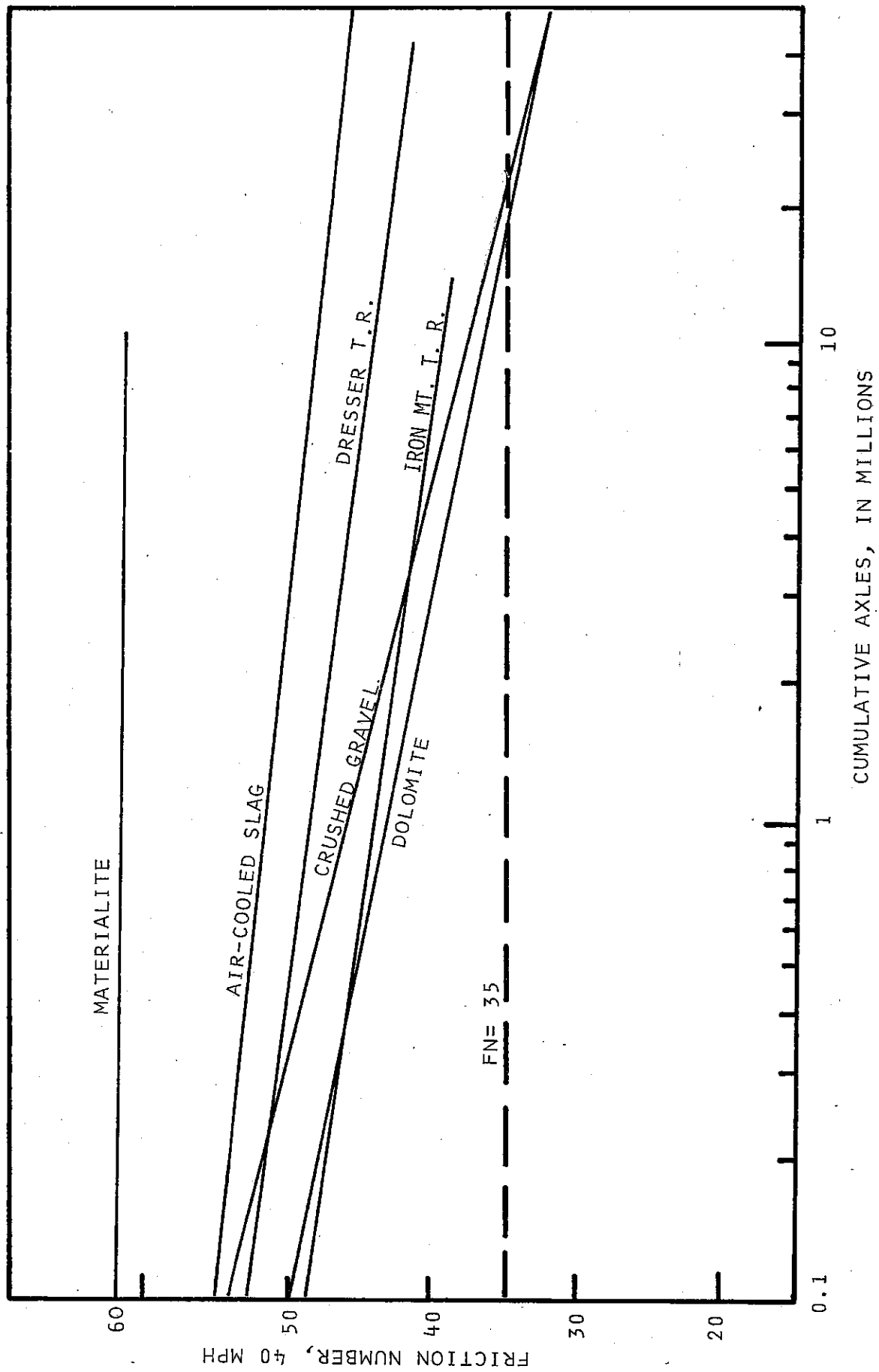


Figure 18. Wear curves for open friction courses (11).

Even though the initial friction number averaged 66, the second measurement approximately 1 year later was 49, a drop of 17 friction numbers. Whether friction will continue to drop was considered to be unknown.

Materialite

Materialite surfaces, most of which had been in service for 3 years, exhibited major distress. Of the 13 sections surveyed, 10 contained major distress, 1 had minor distress, and only 2 had minute or no distress. The surface placed at the signalized intersection of Route Illinois 83 and St. Charles Road, for example, disintegrated soon after it was placed. The mode of failure was a structural loss in the form of raveling and delamination. This distress was associated with high traffic volumes having a high percentage of trucks making turning movements, which caused the low-crushing-strength aggregate to ravel. A similar failure occurred on Route Illinois 47 at Mazon Road. All but 3 of the sites exhibited major total overall distress. Although most of the raveling was associated with places where turning movements occur, 4 sections involved bridge decks where water was trapped by the dams at longitudinal and expansion joints, and 1 section involved the low side of a superelevated curve. In addition to area raveling and delamination, Materialite exhibited minor popout and edge raveling, which was attributed to snowplow abrasion as well as traffic. These raveled areas leave an unsightly, poor riding surface over which the public must travel. Because Materialite was observed to lack durability and to ravel easily even under light traffic, it was considered to be unacceptable for open friction courses. Its use has been discontinued.

Crushed Gravel

Crushed gravel had been in place for 3 years at the time of the survey and was limited to 4 sections. One section on Route Illinois 53 at Naperville Road in Romeoville was replaced in 1978 with a dense-graded slag mix. The other 3 were experimental sections on I-55: northbound and southbound at California Avenue, and northbound south of Route Illinois 53. Both sections at California Avenue had major area and major wheelpath distress while the section south of Route Illinois 53 had only minor area distress. Structural distress ranged from major edge and area raveling to minor joint and popout raveling as well as minor delaminations. Edge and longitudinal joint raveling was associated mainly with snowplow abrasion.

The crushed gravel sections had lost much of their permeability because of a closing of the surface; however, they still maintained some surface drainage because traffic had dislodged individual aggregates adding texture.

Experience, although limited to 4 sections, was seen not to be encouraging. On the basis of 3 years experience, a projected service life of more than 5 years was considered questionable. Crushed gravel was providing friction numbers in the low 30's.

Dolomite

Dolomite sections, in service from 2 to 3 years, were observed to exhibit major distress in wheelpaths and major overall distress in the form of both lost permeability and lost structural integrity. The observed sections had either a partly closed or a closed surface and evidence of both minor and major wheelpath flushing. Small fat

spots also were noticed in three sections. Even though most of the sections had lost permeability because of a closing of the surface, the dolomite mixtures, like the crushed gravel mixtures, continued to maintain some surface drainage because of a number of individual aggregates being dislodged from the surface.

As for structural loss, all types of raveling and some delamination were evident. Snowplow abrasion was evident along edges and longitudinal joints as well as between wheelpaths. Evidence of major and minor popout raveling as well as minor and major area raveling and delamination were noted.

The loss of large areas of surface on US 150 at Downs and at LeRoy was attributed to the percent fines in the mix being reduced to the low limit of the allowable range (5%-15%), which correspondingly increased the percent of voids. A higher void content increases the water-holding capacity of the surface, making it more susceptible to stripping and to freeze-thaw damage. Moreover, some doubt was seen to exist about whether a heat-stable antistripping agent was used by the contractor. The same mix with 14.7 percent fines (near the high limit of the allowable range) placed on US 51 north of I-55, contained only minor overall distress, mainly snowplow abrasion along the edges and between the wheelpaths.

After 2 to 3 years service, snowplow abrasion and frequent popouts were very noticeable in the dolomite sections. With distress progression at the present rate, a projected service life beyond 5 years was seen to be questionable.

Dolomite surfaces were providing pavement friction in the low 30's.

Experimental Sites

As previously mentioned, several experimental sections had been constructed and placed under observation at four sites prior to the 1979 survey. Traprock and Materialite were being evaluated on South Shore Drive in Chicago. Crushed gravel, dolomite and air-cooled slag were being monitored on I-55 at California Avenue and south of Route Illinois 53, while air-cooled slag, steel slag, Materialite, and traprock were being compared on Route Illinois 143 between Wood River and Edwardsville. The discussion that follows identifies observed differences in performance in the experimental OGAFS sections at the time of the survey.

South Shore Drive.--At this site, a city street in Chicago (ADT 7,100), the Materialite surface had almost disappeared whereas the traprock was structurally sound but has reduced permeability. The high asphalt content in the traprock had caused a closed surface and minor wheelpath flushing. Many fat spots also were observed.

I-55, California Avenue.--This site, a six-lane divided urban expressway in Chicago (ADT 130,000), contains air-cooled slag, dolomite, and crushed gravel, respectively, in the outer, middle, and inner northbound lanes and crushed gravel, dolomite, and air-cooled slag, respectively, in the outer, Middle, and inner southbound lanes. Truck traffic is restricted to the outer and middle traffic lanes while only passenger cars travel on the inner traffic lane. Observations suggested that a greater number of heavy trucks travel northbound at this point than southbound.

The air-cooled slag section in the northbound lane was raveled more than that in the southbound lane. The difference was attributed to truck traffic not being permitted in the southbound inner lane. A corresponding

difference had occurred in pavement friction. The northbound pavement friction in 1978 averaged 32 as compared with 44 southbound.

In the crushed-gravel sections, the outer southbound lane, on which trucks travel, showed more area raveling than the northbound inner lane, which carries only passenger cars. The thermoplastic edgeline along the northbound inner lane also was almost completely gone, removing with it the underlying OGAFC surface.

The pavement friction number southbound in 1978 was 29 as compared to 34 northbound. The higher northbound friction was attributed to the absence of truck traffic.

In the dolomite sections, the main difference occurred in raveling. The northbound lane, where truck traffic appears heavier, had more frequent popout raveling than the southbound lane, but the southbound lane had major area raveling as compared with minor area raveling in the northbound lanes.

The pavement friction number was 29 southbound and 32 northbound.

After 3 years service under an ADT of 130,000, the southbound air-cooled slag was performing best. The northbound air-cooled slag section was performing poorly because heavy trucks were abrading the vesicular slag. A projected service life for the northbound air-cooled slag, the southbound crushed gravel, and the northbound and southbound dolomite beyond 5 years was seen to be questionable. A service life of more than 5 years was projected for the northbound crushed gravel and the southbound air-cooled slag sections.

I-55 South of Route Illinois 53.--This site, in the northbound lanes of a four-lane divided rural Interstate (ADT 28,900), begins with a section of air-cooled slag followed by a section of crushed gravel and a section

of dolomite. Overall, the slag had only minute distress while the crushed gravel and crushed dolomite, respectively, contained minor and major distress when surveyed. Snowplow abrasion was evident in all sections but the degree of distress ranged from minute in slag to major in dolomite. The degree of distress appeared to be related to the hardness of the aggregate. After 3 years service, a projected service life beyond 5 years was seen to look good for slag but questionable for crushed gravel and dolomite.

As for permeability, the slag surface was open and the crushed gravel and the dolomite surfaces were closed. Nevertheless, all sections still maintained some surface drainage because individual aggregates had become dislodged from the surface, providing drainage.

Route Illinois 143 between Edwardsville and Wood River.--This site, consisting of a two-lane rural primary highway (ADT 8,500), provided a comparison between Materialite, Iron Mountain traprock, air-cooled slag and steel slag. Overall, Materialite was the only section displaying major distress; the other 3 sections showed minute or no distress.

Materialite had lost some permeability and contained evidence of raveling. Permeability was restricted by a partly closed surface and by minor flushing in the wheelpaths. The structural loss was major area raveling and minor edge raveling. Of the 4 sections, Materialite had the highest average friction number, 58.

Both air-cooled and steel slag had partly closed surfaces and minor wheelpath flushing. The steel slag section, however, contained a few fat spots. These factors tended to reduce permeability in the slag sections. As for structural distress, occasional popout raveling had occurred in

the air-cooled slag; otherwise, structural distress was not evident. The average friction number for steel and air-cooled slag, respectively, was 49 and 46.

Traprock had minor flushing like that of the slags, and a few fat spots with an associated decrease in permeability. The only structural distress was some minor area raveling. Of the 4 sections, traprock had the lowest average friction number, 38.

A general loss of permeability in all test sections was attributed to asphalt contents not being adjusted to take into account the specific gravities of the various aggregates.

These sections had been in service 4 years. If the structural distress that had occurred thus far was to continue at its present rate, a service life up to 10 years for the slags and traprock was believed likely, but a service life beyond 5 to 6 years for Materialite was believed questionable.

Distress Mechanisms

Of the two distress modes catalogued in the survey, structural loss was found to predominate over loss of permeability. A number of factors were observed to contribute to the observed distress. The interactions that take place were seen to be complex, and the individual factors were not easily identifiable.

This section records an attempt by the survey team to relate observed distresses to geometric design, mix design, construction procedures, and maintenance practices.

Geometric Design.--Geometric design seemed to have a profound influence on the performance of an OGAF. Distress was more likely to occur on high-

volume, slow-speed urban streets than on high-speed rural highways. Intersections, where braking and heavy turning movements occur, and vertical sag curves, bridge decks, and the low side of horizontal curves, where drainage was poor, had a greater risk of becoming distressed than straight, open highways.

OGAFC's placed directly on PCC pavements appeared prone to develop delaminations. The delaminations usually occurred near joints, along cracks, and in sag curves where water tends to collect. To assure a longer service life, the OGAFC should be placed on an intermediate bituminous leveling course, which itself is placed on a structurally sound base.

Mix Design and Aggregate Gradation.--Asphalt contents should be adjusted according to traffic volume characteristics, site characteristics, and aggregate specific gravity. Excessive asphalt causes surface closure and wheel-path flushing, which reduces permeability and which may lower friction, depending on the degree of surface closure.

Gradation of the aggregate also influences OGAFC performance. When the percent fines passing the No. 8 sieve exceeds 15 percent, the void content is reduced, resulting in a loss of permeability. On the other hand, if the percent fines passing the No. 8 sieve is near the lower range limit of 5 percent, the void content is increased, thereby adding water-holding capacity and promoting stripping and freeze-thaw damage. This was most evident in the dolomite sections in LeRoy and in Downs where severe raveling had occurred as a result of lowering the quantity of fine aggregate.

Aggregate Characteristics.--Aggregate characteristics influencing OGAFC performance are: friction, hardness, abrasion, crushing strength,

absorption, stripping, and weathering. Soft aggregates that abrade and polish easily and that have a low crushing strength usually develop raveling sooner than hard, polish-resistant, non-stripping aggregates, comprised of both hard and soft minerals. Stripping aggregates are more likely to develop popout raveling, which ultimately develops into area raveling. Low crushing-strength aggregates like Materialite, on the other hand, develop raveling when they are exposed to heavy truck traffic. Highly absorptive aggregates may cause within-mixture asphalt content variations. When this occurs, the lean part of the surface becomes susceptible to raveling, and the rich part reduces permeability.

Construction Procedures.--After the mix design is formulated, careful attention must be given to the construction phase to assure satisfactory OGAFc performance.

At the plant, several factors must be considered to assure a well-performing surface. First, the plant operator, before starting mix production, must clear the plant's aggregate bins so that the mix is not contaminated with a different aggregate. Whether this is a serious problem depends on the amount and the type of aggregate that remains in the bin. An example of aggregate contamination was observed at the start of one traprock job.

Second, the operator must be sure that the aggregate is completely dried before adding the asphalt to reduce the potential for stripping. Highly absorptive aggregates may sometimes need double-drying, depending on the production capacity of the mixing plant.

Third, if an anti-stripping agent is required, it must be heat stable to assure its effectiveness.

Finally, the operator should not let the mixing temperature rise more than 10 to 15 degrees above the target mixing temperature to control asphalt drain-down during transit to the job. The survey indicated that fat spots were most likely to occur at the beginning of OGAFc jobs. This probably resulted from the batch truck standing too long before placing the material. Limitations should be established for hauling and standing times.

Adequate preparation of the underlying surface also is essential to satisfactory OGAFc performance. The pavement on which the OGAFc is to be placed should be structurally sound. An OGAFc cannot be expected to correct pavement deficiencies. After minor base failures have been repaired, a leveling course should be placed to remove surface irregularities and channeling, which can prevent efficient removal of surface water through the OGAFc. Moreover, trapped water contributes to delamination, stripping, and freeze-thaw damage. Placing an OGAFc on a new leveling course enhances bond as well as internal drainage.

As the spreader places the mix, roller operators must set the surface close behind the paver before it cools. Over-rolling after the surface has cooled may result in fracture of the asphalt binder, promoting raveling under traffic. This situation is more critical during cool spring and fall days than during warm summer days.

Maintenance Practices.--After an OGAFc is placed and opened to traffic, it is affected not only by traffic but also by highway maintenance practices.

Maintenance crews need to be especially watchful that drainage of an OGAFc is not obstructed in poor drainage areas. If water flow is blocked, the risk of surface delamination and aggregate stripping is increased.

Surface repairs in distressed areas are very difficult. When they are made in poorly drained areas, they sometimes may accelerate distress. For example, repairs made along pavement edges with a sand mix act as a dam retaining water in the OGAFc, promoting further delamination and increasing the risk of stripping.

Some sites had thermoplastic pavement markings placed directly on the OGAFc surface. A number of edgelines as well as the underlying OGAFc were gone. Apparently, the bond between the thermoplastic and the aggregate was greater than that between the OGAFc and the underlying pavement. This loss was observed in Materialite, air-cooled slag, crushed gravel and dolomite surfaces. Placing thermoplastic edge lines just outside the OGAFc surface, as has been done in several sections, may lengthen the service life of the pavement marking as well as the OGAFc.

Evidence of snowplow abrasion along edges, longitudinal joints, and high spots was found at several locations regardless of the type of aggregate used. Abrasive damage from snowplows can reduce the effective service life of OGAFc surfaces. Snowplow damage was more frequent in the Materialite, crushed gravel, and dolomite sections than in the traprock and air-cooled slag sections. The traprock and air-cooled slag aggregates appear to have greater resistance to snowplow damage than crushed gravel or dolomite.

Service Life

As of 1979, OGAFc's had been in service from 1 to 5 years. Traprock and air-cooled slag aggregates had provided satisfactory service longer than other aggregates used. They were seen to show promise for providing a service life of 10 years. Steel slag surfaces also were believed to provide a service life of 10 years, if the performance of the single site constructed to date.

proves representative. Steel slag has performed well in dense-graded mixes on the Illinois Tollway as well as in Canada and in England.

Novaculite, except for one 3-year-old site, had been in service for only 1 to 2 years. So far, these sections had provided excellent drainage and friction. Although none of the sites had exhibited major overall distress, the amount of major area raveling and crack raveling as well as the amount of minor popout and edge raveling that had occurred within 2 years were seen to suggest a service life beyond 5 years to be questionable.

Crushed gravel and dolomitic sites had been in service for 3 years, and a majority of the sites displayed major overall distress. The survey indicated a loss of both structural integrity and permeability. Moreover, these sections had relatively low friction levels. Under the best of conditions, a service life beyond 5 years was viewed as unlikely. At the end of 3 years they were functioning more like dense-graded than open-graded surfaces.

Materialite, because of its low crushing strength and its high abrasion, had given the shortest service life of any of the aggregates in service.

In summary, air-cooled slag, traprock, and possibly steel slag were seen to have the best chance of reaching a 10-year service life. The service life of novaculite at this time was viewed as questionable, experience so far suggesting 5 to 6 years as a maximum service life. After only 3 years of service, the crushed gravel and the dolomite sections had lost most of their permeability and were experiencing structural distress. Their projected service life was estimated to be less than 5 years.

Cost

Cost comparisons for typical dense-graded bituminous concrete overlay systems and an OGAFRC system as estimated in February 1980 are shown in

Table 12 on page 76. Because the OGAFc surfacing is not considered a structural member of the system, its 5/8-in. thickness becomes an addition to the total 3-in. thicknesses shown for the dense-graded mixtures that are full structural systems. Although some savings can be realized by using a full 3-in.-thick binder course under the OGAFc rather than a combination of binder and surface courses, the added cost of the OGAFc still is substantial.

Recommendations

Analyses of the results of the May 1979 survey of the condition of the 52 OGAFc surfaces constructed in Illinois during the years 1974-1979 led to the following set of recommendations. Except for the final recommendation, all are necessarily tentative because of the short service periods of the pavements examined and sometimes the use of improper construction procedures due to inexperience.

- (1) Limit the use of OGAFc to high-accident sites.
- (2) Continue the use of crushed air-cooled slag and crushed traprock as coarse aggregates in OGAFc.
- (3) Prohibit the use of crushed dolomite, crushed gravel and Materialite as coarse aggregates in OGAFc.
- (4) Avoid placing OGAFc in areas where surface drainage is poor.
- (5) Avoid placing OGAFc on bridge decks and other places where drainage is obstructed.
- (6) Avoid placing OGAFc directly on PCC pavement, even after priming the surface.
- (7) Continue experimenting with other aggregates that appear to offer potential as satisfactory aggregates for OGAFc.

Interim Evaluation of Skid-Accident Reduction, OGAFC Project

An evaluation of the effect of one open-graded asphalt friction course in reducing skid accidents is being made on Interstate Business Loop 55 (Sixth Street) in Springfield from Stevenson Drive to Stanford Avenue. Awarded in May 1977 at a cost of \$179,417, the project included a 1 1/2-in. bituminous concrete binder course with an overlay of 5/8-in. open-graded asphalt friction surface course using crushed traprock coarse aggregate. The total length of the project was 0.652 mile. Construction was completed September 14, 1977. No physical defects were noted in the general OGAFC condition survey of 1979.

The improvement is in a moderately commercialized area with plant entrances to the East and several small business establishments to the West. The location was selected on the basis of having had several high-accident rate locations with an average total of 93 accidents per year over the four-year period 1973 through 1976. Approximately 32 accidents per year, or 34 percent of the annual average total, occurred on wet pavement. Friction tests conducted prior to the improvement produced friction numbers ranging from 17 to 30, with an average of approximately 22.

The following table shows the overall accident experience that has occurred at the improvement location during the past six years, 1973 through 1978. A final assessment will be made when two years of "after" data have been analyzed.

<u>Year</u>	<u>ADT</u>	<u>Total Accidents</u>	<u>Property Damage Only</u>	<u>Personal Injury Accidents</u>	<u>Number Injured</u>	<u>Fatal Accidents</u>	<u>Number Killed</u>
1973	31250	88	67	21	27	0	0
1974	31250	78	65	13	20	0	0
1975	34050	111	83	28	44	0	0
1976	34050	94	62	32	54	0	0
1977	35300	68	48	20	28	0	0
1978	35300	59	42	17	27	0	0

Further tabulation of the data shows the occurrence of accidents on wet pavement as follows:

<u>Year</u>	<u>Total</u>	<u>Property Damage Only</u>	<u>Personal Injury Accidents</u>	<u>Number Injured</u>	<u>Fatal Accidents</u>	<u>Number Killed</u>	<u>% Wet Pavt. Acc. are of Total</u>
1973	29	25	4	6	0	0	33.0
1974	33	28	5	6	0	0	42.3
1975	37	27	10	16	0	0	33.3
1976	27	19	8	15	0	0	28.7
1977	15	10	5	6	0	0	22.1
1978	9	7	2	3	0	0	15.3

The percent that accidents occurring on wet pavement have to the total indicate a reduction after improvement. Analysis of the specific period of September 15, 1977 to December 31, 1978, and comparisons with the "before" period from January 1, 1973 to December 31, 1974, show a definite effect by the improvement. These comparisons are given below.

	<u>Period</u>	<u>Total Accidents</u>	<u>Wet-Pavement Accidents</u>	<u>Percent Wet-Pavt. Accts. are of Total</u>
Before (24 mo.)	1/1/73 - 12/31/74	161	62	38.5
After (15½ mo.)	9/15/77 - 12/31/78	77	9	11.7

NOTE: Accident data for 1975 and 1976 are not included because other improvements were made at the location in those years. Those intermediate improvements consisted of buried left-turn lanes at two locations and upgrading the traffic signals at four major intersections within the section. Improvements such as these could combine to have an effect on the accident experience through the location.

The above table shows that the percentage of wet-pavement accidents to total accidents reduced 26.8 percent since completion of the skid treatment. Local climatological data for "before" and "after" study periods indicate no significant difference in the number of precipitation days (rain or snow) conducive to skidding-type accidents. Precipitation days during the "before" period totaled 242 or 33.2 percent, and for the "after" period, 162 or 34.3 percent.

As mentioned earlier, the friction numbers at the location were found to be low when tested in 1972 and 1974. The friction numbers at Stanford Avenue averaged 24 while the remaining section had numbers ranging from 17 to 30. Following the paving, friction tests were made in 1977 and 1978. Results of those tests are given below.

<u>Date Tested</u>	<u>Direction</u>	<u>Friction Number (FN₄₀)</u>		
		<u>Outer Lane</u>	<u>Middle Lane</u>	<u>Inner Lane</u>
10/24/77	Northbound	46	47	45
	Southbound	45	45	43
10/11/78	Northbound	39	40	40
	Southbound	37	38	39

Correlation between friction numbers and skidding-accident experience will be made at the end of the two-year "after" period.

Statistics available to date indicate that the new friction course has been effective in reducing accidents occurring on wet pavement. The proportion of wet-pavement accidents to total accidents shows a significant reduction of 26.8 percent (95 percent level of confidence, 1-tail test) from the "before" to the "after" improvement period. Wet-pavement accident severity also has reduced since friction treatment. The rate of personal injury accidents for the section declined 70.5 percent from 61 to 18 accidents per million vehicle miles.

Friction-trailer test results show an average wear rate of 14 percent per year with friction numbers measured from 38 to 40 in October of 1978. Additional friction-trailer tests, on-site inspections, and accident records review will be included in a final assessment of the friction treatment. That report will contain two full years of "after" improvement data plus benefit/cost computations to determine the relative safety payoff of the friction treatment.

SAND-ASPHALT MIXTURES

As indicated earlier, sand-asphalt mixtures in which the maximum aggregate size is in the No. 4 sieve (3/16 in.) or 1/4-in. range frequently can be advantageous as a resurfacing material where a reasonably sound pavement structure already exists and all that is needed is a thin layer of material to upgrade low surface friction or lack of smoothness. Although larger aggregate particles generally are recognized to be needed to provide the necessary macrotexture for safe high-speed travel, the sand-asphalt mixtures that can be placed in layers of no more than 1/2-in. thickness appear to have the potential for providing adequate skid resistance for travel at speeds ranging up to perhaps 40 mph. A summary description of experimentation that Illinois has entered into to make best use of these mixtures follows.

Plasticized Bitumen Hot-Mix Seal

Plasticized bitumen hot-mix seal, as noted earlier in the chapter, consists of a specific blend of stone screenings and sand, plus asphalt and an additive known as Tapisable. Another additive identified as Recopave also is acceptable. Specifications covering current use of the material are given in Appendix C. The plasticized bitumen hot-mix seal has been the subject of a considerable amount of experimentation over the past several years in Illinois.

The plasticized bitumen hot-mix seal was first used in Illinois in city work in Aurora and Champaign. The first application was made in Aurora in August 1966 where a 3/4- to 1-in. overlay was placed on a residential street using a motor grader equipped with a special spreader box. The first application in Champaign was an overlay of nominal 5/8-in. thickness placed in May 1967 with a conventional paver in short sections on residential streets. Friction tests made two to three years after construction showed FN_{40} values ranging from the 40's to the 60's, and still above 40 two years later.

The first installation using State funds involved placing the material at a nominal 3/4-in. thickness on several streets in Aurora during the period 1970-72. All of the streets had ADT volumes below 5,000. Some experimental variations in construction procedures were subjected to special observation during the construction process, and performance evaluations were made for several years following construction.

Three different methods of placing the mixture were compared during the study: standard machine laying; laying with a Layton track paver; and a combination of windrow spreader and a motor patrol grader equipped with a special spreader box. The motor grader with special attachments was considered to have an economic advantage where smoothness of surface and joints might not be of prime importance, but the conventional paver produced the smoothest surface. No tack coat was needed for binding the full mixture to the old surface. Omission of the additive over a short stretch showed loss of workability and the need for a tack coat without it.

The results of friction tests made over a period of several years on the earliest plasticized bitumen surfacing placed in Aurora in 1966 and

on seven of the street surfacings placed in 1970 are shown in Table 16 along with sample daily traffic counts. It will be noted in the table that, with the exception of the W. Indian Trail surfacing that was extremely fat due to excess crack filler, all surfaces maintained FN_{40} values in the satisfactory range. Traffic speeds were not high on the city streets. All surfaces showed good stability under traffic.

Based on the uniformly good service being given by the installations in place in 1972, as indicated both by friction testing and observations of physical stability, a specification for the new plasticized bitumen hot-mix seal was prepared and its use approved for State-funded work on two-lane urban streets where design ADT volumes did not exceed 5,000. Many miles of the surfacing were placed under the specification with good results.

Continued all-around good service being provided by the plasticized bitumen hot-mix seal under low-traffic, low-speed conditions led to further experimentation under higher traffic volumes and speeds. The first trial installation under more severe conditions was placed in November 1973 on Route Illinois 83 immediately south of the Cal-Sag channel bridge in Cook County. This is a four-lane pavement with an ADT volume of 19,000, including 3,200 commercial vehicles, of which 950 are multi-unit trucks. A variety of geometric situations, including a horizontal curve, a vertical curve, and a signalized intersection exist at the location. Although the friction numbers recorded for the surfacing from the first year onward have not been as high as recorded in most of the previous work, they compare favorably with those often recorded for dense-graded bituminous concrete under similar conditions of exposure. Observed minor shoving on the section suggests that

TABLE 16

FRICTION NUMBERS FOR TRIAL INSTALLATIONS OF PLASTICIZED BITUMEN HOT-MIX SEAL (WITH TAPISABLE ADDITIVE) IN AURORA (Adjusted to 40 mph)

Location	Const. Date	Average Daily Traffic	Direction of Travel	Date of Test and Friction Numbers									
				10/69	12/70	05/71	07/72	09/73	10/74	09/75	10/76		
Rosedale Ave.	Aug. 1966		N S	55 53	63 62	62 63	56 61	59 54	50 51	57 58	50 53		
Buell Ave.	Oct. 1970	375	N S		62 62	64 62	61 62	62 65					
Westlawn Ave.	Oct. 1970	950	N S		60 58	64 64	61 60	62 64	53 47	62 58	62 62		
California Ave.	Oct. 1970	1,675	E W		59 63	59 60	50 54	47 55	41 40	52 51	56 56		
Florida Ave.	Oct. 1970	1,850	E W		55 61	51 53	41 45	37 43	28 29	43 45	46 44		
W. Indian Trail	Oct. 1970	2,875	E W		62 55	49 47	33 35	32 40	32 27	34 37	32 35		
Elmwood Drive	Oct. 1970	3,600	N S		53 52	46 43	37 40	42 41					
Palace St.	Oct. 1970	4,800	N S		55 55	40 42	39 38	40 39	30 32	39 44	43 44		

a deficiency in stone screenings with a corresponding excess of natural sand may be responsible for the reduced, although generally satisfactory, frictional performance. Following are the results of friction tests recorded on the section during the first 6 1/2 years following construction in November 1973:

<u>Date Tested</u>	<u>Northbound Lanes</u>		<u>Southbound Lanes</u>	
	<u>Outside</u>	<u>Inside</u>	<u>Outside</u>	<u>Inside</u>
Oct. 1974	35	28	26	35
Sept. 1975	39	37	36	38
Nov. 1976	44	32	28	46
Oct. 1978	38	35	32	37
June 1980	38	36	35	40

Continued good performance of plasticized bitumen surfacings led to the extension of use to two-lane rural pavements and to use under design ADT volumes up to 10,000.

Further experience under the higher traffic volumes and under higher traffic speeds has been accumulating and is to be covered by a report on which preparation is expected to begin in the immediate future.

The plasticized bitumen hot-mix seal mixtures currently cost about \$6 per ton more than the more coarsely graded mixtures that have long been in use; however, with layer thicknesses of no more than 5/8 in. practicable, their use has become attractive where a reasonably stable substructure already exists.

Stone Sand

The known angularity of stone sand (as produced by washing or air separation in crushing operations) led to its selection for trial use at four locations as a potential aid to skid resistance as the primary aggregate in sand-asphalt mixtures. Two of the test locations were in the

Chicago area, on 25th Avenue at Eisenhower and on Cermak Road at 25th Avenue, one was on Route US 66 at Odell Road, and one was in the Troy experiment described previously in this chapter. The Cermak Road experimental mixture contained a rubber additive, which observations eventually indicated to have no appreciable effect. The locations in Chicago were four-lane roadways with ADT volumes of 23,000, that at Odell was a four-lane roadway with an ADT volume of 13,000, and that at Troy was a two-lane pavement with an ADT volume of 2,200.

All of the sands were manufactured from limestone nominally passing a No. 4 sieve. The Chicago and Odell installations were made in 1969 and 1970; that at Troy was placed in 1971.

Measured FN_{40} values were mostly in the established satisfactory range, with the remainder in the marginal range, when first tested following construction at the four locations. The Troy location showed no values at the satisfactory level. Later measurements one year and two years after construction showed FN_{40} values all at the marginal and low levels. The results of the study were concluded to indicate that the limestone sands tested had no special capabilities for enhancing skid resistance and that no further experimentation was warranted.

Natural Sand

Natural sand was used as the primary aggregate in sand-asphalt mixtures placed experimentally at two locations in Urbana, one in 1969 and the other in 1970, where thin surfacings were desired. The first experiment included surfacings placed on Pennsylvania Avenue and Crystal Lake Drive in Urbana. Both streets had 25 mph speed limits and low traffic volumes. A rounded-

particle sand (Illinois Type A) with about 5 percent retained on the No. 4 sieve was used in the mixture.

Friction tests made in 1969, 1970, and 1971 produced FN_{40} values in the 50's, well above the established minimum for the satisfactory range. The number of axles passing over the two sites was relatively low, not exceeding 700,000 in the nearly three years of use.

Natural sand also was used as the primary aggregate in a bituminous mix placed experimentally on Route US 40 east of Effingham in 1970. Route US 40 at the location is a two-lane rural highway with an ADT volume of 5,000. The mix containing the rounded sand was placed in a nominal 1/2-in.-thick overlay. A regular Class I surface course mixture of 1/2-in. nominal top size aggregate placed at the same time to a nominal thickness of 1 1/2 in. was available for comparative friction tests.

When friction tests of the two mixtures were made in September after the passage of about 150,000 axles following placing in August, FN_{40} values for the sand mix were in the low range slightly below the marginal level, and those for the Class I mix were in the marginal range. Friction tests made in June 1971 showed all friction numbers to be in the low range. Additional tests made in October 1971 after the passage of about 2,300,000 axles over the sand mix and 1,800,000 over the Class I mix produced friction numbers in the marginal range for all sections. Little difference was seen in the frictional performance of the two mixes.

The experimentation with natural rounded sand as the primary aggregate in sand-asphalt mixtures was concluded to suggest that the rounded sand imparted no special frictional properties to the mixture and that further experimentation was not warranted.

Slag Sand (Air-Cooled Slag)

The experiment on Route US 40 near Troy that has been mentioned previously in this chapter in connection with slag coarse aggregate mixtures also contained a section having a mixture in which a sand made from air-cooled blast furnace slag was the primary aggregate. Pertinent information relative to this mixture and the results of friction tests appears in that portion of the text and in Tables 6, 7, 8, and 9.

The friction tests conducted yearly over a six-year period following construction may be noted to have shown FN_{40} values ranging between 38 and 59, all within the established satisfactory range. It will be noted, though, that friction number-speed gradient derivations from friction numbers measured at 30 mph and 50 mph were in a range that would suggest confining use of the mixture to locations of low- to moderate-speed travel. The larger quantity of coarser material (retained on the No. 10 sieve) in the air-cooled slag mixture as compared with that fraction in all other sands used experimentally (40 percent as compared with 18 percent at the most) is likely to be responsible in part for the relatively high friction numbers of the mixtures containing the slag material.

It appears from the results of this single experiment that further experimentation and use of a sand-asphalt mix containing air-cooled blast furnace slag sand similar to that tested as the primary aggregate at Troy may be appropriate where a thin layer of material is needed to improve skid resistance and where speeds over 40 mph are not expected.

Traprock Sand

Traprock sand, a crushed igneous stone by-product of the extraction of metal ore, was used experimentally in sand-asphalt mixtures at two

locations, one of which included the resurfacing of three intersections on Route US 36 (Eldorado Street) in downtown Decatur. Eldorado Street at the location is a four-lane street with ADT volumes varying from 20,000 to 27,000 and a speed limit of 25 mph. The mixture was placed to a 1/2-in. thickness at approaches to the signalized intersections, varying in length from 140 to 400 feet.

The traprock aggregate, which was obtained from Iron Mountain, Mo., was of sand size, with almost two thirds lying between the No. 10 and No. 40 sieves. To improve the stability of the mixture, a natural sand was added to increase the finer aggregate fraction.

Friction tests made in September 1969 after construction in June, and 1/2 to 1 1/2 million axle passes, showed average FN_{40} values between 27 and 38. During four additional readings through December 1971, at which time about 9 to 15 million axle passes had accumulated in the traffic lanes and 6 to 10 million in the passing lanes, FN_{40} values ranged between 29 and 43. No downward trend in friction numbers that might be attributed to wear was observed.

The second of the two locations at which traprock sand was used in a sand-asphalt mixture was on Route US 66 at a signalized intersection with Fifth Street near Lincoln. Route US 66 at the location was a four-lane divided highway with an ADT of 13,000 and a speed limit of 45 mph. A liquid rubber, R-504 manufactured by Firestone Tire and Rubber Company, was added to the mixture in the amount of 0.3 percent by weight of solids in the mix. The traprock aggregate was obtained from the same source as that of the traprock aggregate in the Decatur experiment. However, in this instance it was of a gradation more typical of that being used in bituminous concretes in Illinois and no blending was done. The mix was

placed to a 3/4-in. thickness in May 1967 and first tested for skid resistance in October 1969.

The initial friction tests made after 10 million axle passes in the traffic lane and 3 million in the passing lane showed average friction numbers of 35 and 44 in the respective lanes. Little change was noted in tests made in 1970 and 1971, except for a sharp loss in the northbound lanes in 1971 that was traced to an asphalt spill. Tests made in June 1972 showed the influence of the contamination to have been removed and average readings to be 30 in the traffic lane after about 23 million axle passes and 40 in the passing lanes after about 8 million passes, indicating a slight downward trend. The rubber additive did not seem to have any appreciable influence on measured surface friction.

It appears from the data assembled in the two tests that traprock sand, when used as the primary aggregate in sand-asphalt mixtures, imparts no extraordinary skid-resistance properties to the mixtures, but can serve adequately for low-speed travel even where traffic volume is fairly high.

Gripstop

This experiment consisted of the use of Gripstop, a modified Kentucky rock-asphalt mix, on Route Illinois 50 at a signalized intersection where Route Illinois 83 is crossed. The installation covered about 500 ft of all four lanes of the north leg of the intersection. Route Illinois 50 at the location had an ADT volume of 21,000.

Gripstop consists of a crushed sandstone containing about 4 1/2 percent of natural bitumen mined in Kentucky that is enriched with additional asphalt before shipment. It is a fairly well-graded material that approaches the specification for the sands used in Class I mixtures.

The mix was laid to a compacted thickness of 1 in. in October 1968 and first tested for friction quality in November 1969. During the year that elapsed between construction and initial testing, traffic lanes were exposed to about 4 million axle passes and passing lanes to about 3 million. FN_{40} values at that time averaged about 38 in the traffic lanes and 44 in the passing lanes. Later readings through October 1971, at which time the traffic lanes had experienced 10 million vehicle passes and the passing lanes about 8 million, remained about the same as the original readings, indicative of a good resistance to polishing. Although the mix retained adequate friction quality throughout the three-year period following construction that friction tests were conducted, the resistance of the surfacing to abrasion under traffic was less than desirable. By November 1971, three years after placement, the surface was worn away completely in the wheelpaths of the traffic lanes.

The results of this single experimental installation suggest that the use of Gripstop may have some application in improving skid resistance in low-traffic-volume situations or where short-term corrective measures are needed in high-volume situations. No further experimentation is currently planned.

Wet-Bottom Boiler Slag

Wet-bottom boiler slag is the hard, angular by-product of coal in wet-bottom boilers. Experimental sand-asphalt mixtures with boiler slag as the primary aggregate were placed at two locations to investigate its frictional properties, the first being placed on the roadway across Spaulding Dam in Springfield in 1966. This material had a nominal top size of 3/16 in. (passing No. 4 sieve) and met the gradation requirements

for fine aggregate for Class I mixtures. The mixture was placed on a two-lane pavement with an ADT of 5,600.

Friction tests made in June 1971 after 9.3 million axle passes produced an average FN_{40} value of 40, within the satisfactory range.

Wet-bottom boiler slag of substantially the same gradation as that used at the experimental site in Springfield also was used experimentally in a sand-asphalt mixture on all four legs of the intersection of Routes US 66 and Illinois 47 near Dwight placed in May 1971. Route US 66 at the location was a four-lane divided highway with an ADT of 14,200; Route Illinois 47 was a two-lane highway with an ADT of 4,800.

Friction tests at the Dwight location made in July 1972 showed FN_{40} values of 30 to 35, all in the established marginal range, where axle passes were 2.4 and 6.4 million. In the passing lanes of Route US 66 with axle passes of 1.7 million, FN_{40} values of 37 and 43 were recorded.

The results of these two experiments appear to suggest that the use of wet-bottom boiler slag as the primary aggregate in sand-bituminous mixtures may be appropriate in low-traffic, low-speed situations.

SPRINKLE TREATMENT

The sprinkle treatment consists of precoating aggregate chips of high frictional quality with asphalt and applying the mixture cold to a hot plant-mix immediately behind the paver where both are then rolled for embedment and compaction. This treatment has been used successfully in England and in several States in this country, including Iowa and Georgia where it is now being used routinely. Its major advantage is one of economy in that it permits the minimum use of a high-friction but costly aggregate to

satisfy frictional requirements where the basic mixture contains a lower quality but less expensive aggregate.

The sprinkle treatment received its first use in Illinois in two trial installations placed in 1980. The following discussion of Illinois experience is based on the preliminary draft of a report describing construction experience and presenting early test data for one of the installations. Experience at both installations is considered to have been similar.

The project reported is on Route Illinois 185 and located immediately west of Route US 40. Route Illinois 185 at the location is a two-lane pavement with an ADT volume of 1,300 to 1,600 but reaching 2,800 in the mile nearest Vandalia. The construction included placing a 3-in. binder course and a 1 1/2-in. surface course plus the sprinkle treatment on 8.8 miles of two-lane pavement in September and October 1980.

The surface course is a typical Class I, Mixture C bituminous concrete with limestone coarse aggregate and meeting Illinois standard specifications except that the amount of material retained on the No. 10 sieve was kept below 50 percent as compared with the usual 60-65 percent to provide space and mortar for chip embedment. The chips were derived from traprock from Iron Mountain, Missouri, an igneous material with a previous record of good frictional resistance. The limestone coarse aggregate used in the bituminous concrete mixture does not have a good record for frictional quality and, as reported earlier, is now used only in light-traffic situations. Typically, the traprock chips were of a gradation in which 66 percent lay between the 3/4-in. and 1/2-in. sieves, 32.5 percent lay between the 1/2-in. and No. 4 sieves, with the remainder passing the No. 4 sieve.

In the construction process, the chips were coated with 1.3 percent of the same asphalt cement as used in the Class I mixture (AC 70-85) at 300°F in a batch plant. Early use of 1.5 percent of asphalt appeared heavy. An antistripping agent in the amount of 0.5 percent by weight of the asphalt was used. The coated chips were stockpiled until use one to nine days later. Spreading of the chips behind the paver was done with a "Bristowes" spreader. As part of the experimentation, three different spreading rates were used: 6 lb/sy; 9 lb/sy; and 12 lb/sy. A two-wheel vibratory roller was used for breakdown and chip embedment. Further rolling was done with a tandem roller. Chip embedment was good except at a few locations where heavy chip application, a thin mat, or perhaps a combination of the two, are believed to have been responsible for the poor embedment.

Friction tests made soon after construction provided values of 35 where the chip application rate was 6 lb/sy; 36 where the application rate was 9 lb/sy; and 38 where the application rate was 12 lb/sy. The relatively low friction numbers appeared to be attributable to a still-remaining asphalt film not yet worn away by tire-aggregate contact, and to a tendency of the hard but fine-grained traprock chips to become oriented during the rolling operation with their smooth flat faces upward and sharp edges out of tire contact.

Texture measurements by the sand-patch method averaged 0.033 in. for the 6 lb/sy chip spread; 0.046 in. for the 9 lb/sy spread; and 0.062 for the 12 lb/sy spread. The average texture depths for the 9 and 12 lb/sy spreads are considered good to excellent.

On the assumption that the service performance of the Class I, Mixture C and sprinkle treatment combination should be comparable to

that of Class I, Mixture D, or Class I, Mixture E, and a 1 1/2-in. thickness of surfacing, relative costs were estimated to be as follows:

Mixture C plus 9 lb/sy of traprock chips - \$2.80; Mixture D - \$2.67; and Mixture E - \$2.89.

The results of the experimentation are considered to indicate that the sprinkle treatment may be a viable, practical, and economical method for providing a safe riding surface if an aggregate of high frictional quality and better microtexture than that characteristic of the traprock is used, including such aggregates as crushed air-cooled slag and crushed-steel slag, and further experimentation using these materials is anticipated.

PORTLAND CEMENT CONCRETE PAVEMENT TEXTURING

In the construction of PCC pavements, the mortar component of the mix, inclusive of the fine aggregate, is floated to the surface where it becomes the principal component subjected to traffic wear. The use of a good quality, wear-resistant fine aggregate, a well-designed concrete mixture, and good construction procedures will do much toward providing acceptable friction quality. However, the friction also is highly dependent on the texture achieved by the final finishing method applied, and can be controlled through selection of an appropriate method.

A major change that was made in the final finishing procedure to improve the friction characteristics of portland cement concrete (PCC) pavements in Illinois was discussed earlier in this chapter. Following is a summary discussion of a field texturing experiment in Illinois that aided in selecting the new final finishing procedure. The discussion is based on an interim report covering only the construction phase and the first series of tests made before the experimental pavement was opened to traffic. A

final report of the study that includes coverage of service performance evaluations is in preparation and scheduled for completion and release in 1981.

I-72 Texturing Experiment

In the Illinois experimentation, seven different textures known from previous experience in Illinois and elsewhere to be among the most promising for producing good friction quality were applied to contiguous sections of pavement during the construction of I-72 east of Springfield in the fall of 1976.

The initial program of monitoring and testing included detailed observations of construction processes and the results achieved, the measurement of friction numbers at different speeds to develop friction number-speed gradients, and measurements of texture depth, internal and external vehicle noise levels, and surface smoothness. Friction tests and other observations made after the pavement was placed in service will be discussed in the report now in preparation.

The seven textures included in the study were formed by: (1) Transverse tining; (2) transverse brooming; (3) dragging an artificial turf longitudinally; (4) transverse rolling with a roller with raised lands; (5) a longitudinal turf drag and transverse tining combination; (6) longitudinal brooming; and (7) longitudinal tining. The most significant observations recorded during construction follow:

Transverse Tining.--The tining mechanism included an 8-ft tine bar mounted on the transverse mechanism centered under the texturing/curing machine. The steel tines were 1/16 in. by 6 in. and mounted on 1/2-in. centers. When only the weight of the tine bar was seen not to apply

sufficient pressure to the tines for obtaining grooves in the plastic concrete, the problem was corrected by the addition of springs. The required minimum groove depth of 1/8 in. was equalled or exceeded after the springs were added. Overlapping of the grooves in succeeding passes caused slight edge damage at the beginning of a traverse. No mortar buildup on the tines or tine wear was observed.

Transverse Brooming.--The transverse broom was the same length as the tine bar (8 ft) and mounted under the texturing/curing machine in the same manner. Overlap of texturing caused some slight edge damage. As with the tining device, no grout buildup or perceptible wear of the broom bristles was observed.

Artificial Turf.--A section of Monsanto Astro-Turf with dimensions of 3 ft by full pavement width minus 2 in. was dragged longitudinally with the pavement to produce the artificial turf texturing. The turf was mounted on a frame across the front of the texturing/curing machine, with a board attached to the turf to provide sufficient weight for adequate texture. No problems were observed in the use of the device.

Transverse Roller.--The transverse roller was 10 ft long with 3/8-in. raised lands (1/8 in. at crest) and 1/4 in. at root on 2-in. centers. A great deal of trouble was encountered in mounting and leveling the roller. Keeping the roller parallel with the pavement for the entire traverse also proved to be a problem. The roller, which was free-wheeling and relied on its own weight for texturing force, at no time produced acceptable grooves, and its use was discontinued before the section planned for its use was completed. It was thought that power rotation of the roller and an anti-stick coating might overcome the difficulties that were experienced.

Artificial Turf and Transverse Tining.--This texturing procedure combined the longitudinal artificial turf drag and transverse tining into a single operation. The texturing was done with the artificial turf mounted on the front frame and the tines mounted on the transverse mechanism of the texturing/curing machine in the same manner as when each was used alone. As when each procedure was used separately, no problems were encountered.

Longitudinal Brooming.--The longitudinal brooming process was the same as the transverse brooming process except that the same type of broom of full pavement width was mounted under the texturing/curing machine and pulled in a longitudinal direction. No problems were encountered in the texturing process.

Longitudinal Tining.--The tining setup for this process was the same as that for the transverse tining process except that the bar was full-pavement width. Some difficulty was experienced in adjusting the height of the tines to obtain the proper texture. Also, it was found that the tines needed to be raised each time the machine stopped to prevent the tines from settling into the plastic concrete.

Friction Test Results.--The results of friction tests made with the ASTM E-274 skid trailer at 30, 40, and 50 mph on each of the test sections before opening to traffic are given in Table 17 and shown graphically in Figure 19. It will be noted in Table 17 that the processes involving transverse texturing, except for the roller process which proved to be the poorest of all, provided the highest friction numbers. It will be noted also that the combination artificial turf and tining process produced the highest FN_{40} values, well into the satisfactory range at the standard test speed. In the past, new PCC pavements textured with a burlap drag

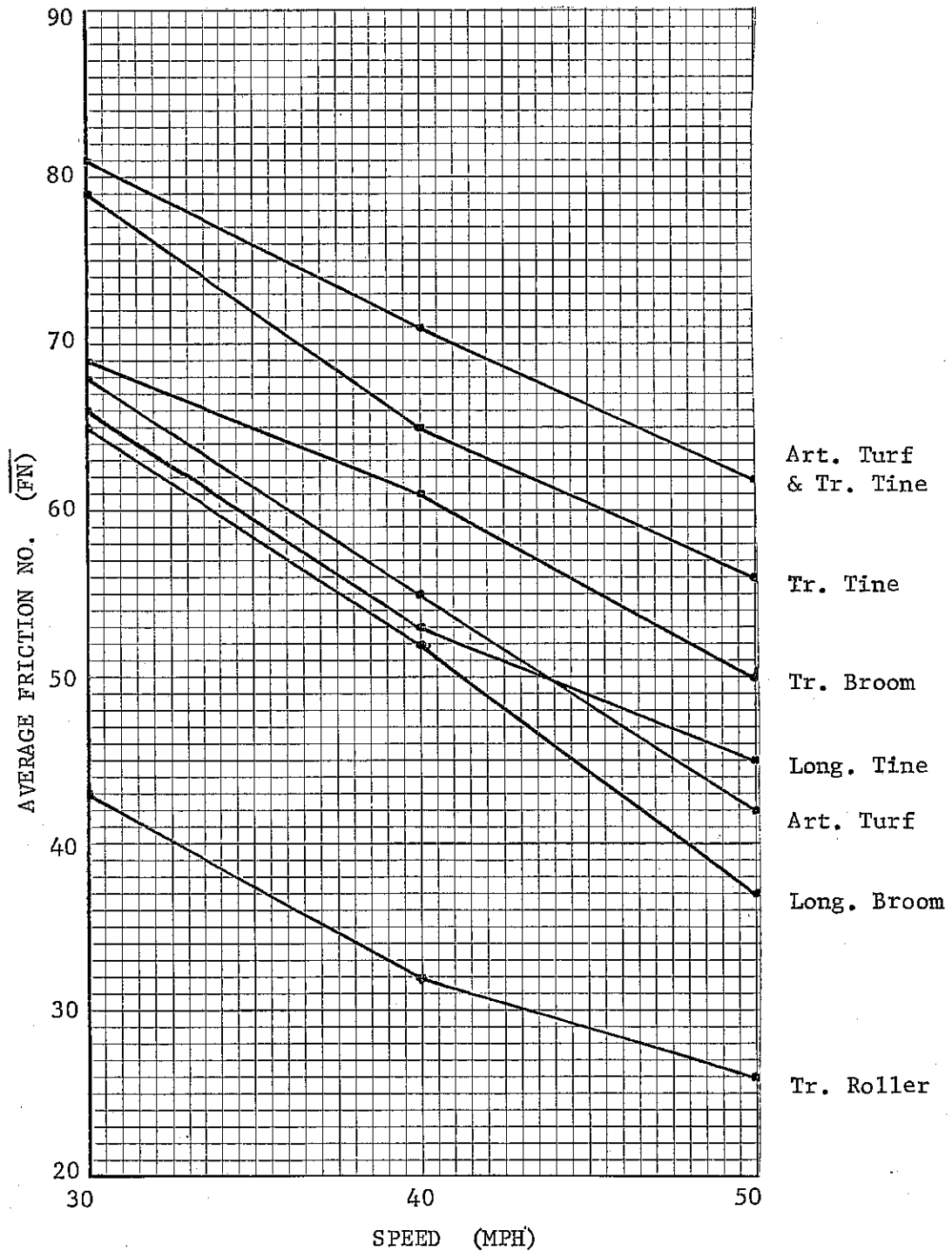
TABLE 17

FRICTION NUMBER AT 30, 40, and 50 MPH (10)

Texture	\overline{FN}_{30}	Range	\overline{FN}_{40}	Range	\overline{FN}_{50}	Range
Artificial Turf & Trans. Tine	81(10)	69-87	71(11)	62-77	62(10)	53-74
Trans. Tine	79(10)	72-86	65(10)	59-73	56(8)	52-61
Trans. Broom	69(10)	60-74	61(10)	56-66	50(8)	45-55
Artificial Turf	68(10)	61-78	55(10)	42-65	42(10)	38-48
Long. Tine	66(9)	60-72	53(10)	46-60	45(10)	39-57
Long. Broom	65(9)	52-76	52(9)	43-65	37(8)	35-41
Trans. Roller	43(10)	40-45	32(9)	30-35	26(9)	23-30

Note: (x) Denotes number of tests.

1.0 mph = 1.609 Km/hr



Note: 1.0 mph = 1.609 Km/hr

Figure 19. Friction Number-Speed Gradient (10).

(previous standard practice) had shown an FN_{40} of about 50, well below those of most of the experimental textures. Somewhat unexpected was the steepness of the friction number-speed gradient curves of Figure 18, based on previous beliefs. Neither the steepness of the slopes, suggestive of lightly developed macrotexture, nor the sameness of the slopes, indicative of little difference in the macrotextures produced by the different texturing processes, seemed to the investigators to be consistent with visually observed differences.

Texture-Depth Measurements.--Texture-depth measurements made on each texture using the sand-patch technique showed only the artificial turf and the artificial turf and tining combination to be above the minimum average depth limitation of 0.03 in. as determined by the sand-patch method that has been used by the Portland Cement Association and the American Concrete Paving Association. Texture-depth measurements by the silicone-putty method gave, in general, appreciably greater average depth values. The texture-depth measurements did not appear to have much value in comparing the different textures.

Noise-Level Tests.--Noise-level measurements were made on the test sections using a station wagon equipped with belted tires and driven at 55 mph. Readings were made inside and outside the vehicle, both under power and coasting. Mean noise levels were all in the 66 dBa to 72 dBa range, a difference insufficient to rule out any texture for further study. As determined inside the vehicle, the order of rank from high to low was transverse tine, transverse roller, transverse broom, longitudinal tine, artificial turf, longitudinal broom, and artificial turf/transverse tine. Subjective observations of the test crew ranked the transverse-roller

texture as the greatest noisemaker, probably because of the humming sound that was produced by the uniform spacing of the roller grooves.

Smoothness Tests.--Roadometer tests by standard procedures produced the following Roughness Index (RI) values:

<u>Texture</u>	<u>RI</u>
Transverse tining	82
Transverse brooming	77
Artificial turf drag	80
Transverse roller	84
Artificial turf/transverse tining	91
Longitudinal brooming	72
Longitudinal tining	77

Adjective ratings of riding quality in the range of the readings are "very smooth" for RI = 75 and less, "smooth" for RI = 76 to 90, and "slightly rough" for RI = 91 to 125. None of the RI values obtained are considered unacceptable for new pavement construction. Differences in the readings were not sufficient to isolate the effects of any of the texturing methods on surface riding quality.

Preliminary Results.--Friction tests indicated that all of the texturing procedures, with the exception of the transverse roller procedure, produced textures that resulted in FN_{40} test values above those to be expected from the formerly specified burlap drag method of texturing. The artificial turf/transverse tining procedure produced the highest FN_{40} values, values that were well above the minimum that is considered satisfactory. Friction number-speed gradients were considered higher than expected for all textures based on visual observations of macrotexture. Texture-depth measurements, noise-level measurements, and smoothness tests did not show strong differences between the textures under observation, and did not show cause to rule any out for further consideration. Overall, the artificial turf/transverse

ting procedure appeared to be the best and the transverse roller procedure the least desirable of those tested. The results were considered to provide confirmation for a decision that had been made to introduce the artificial turf/ting procedure into regular practice. Service Evaluation

Evaluation of the artificial turf/transverse ting texture as now specified and the other textures applied in the experimental construction of 1976 on I-72 has continued and is to be discussed in the report scheduled for completion in 1981. Evaluations also have started on the Edens Expressway in Chicago and at two other locations, but at this early stage have not produced definitive information on what ultimately can be expected from the artificial turf/transverse ting texturing process.

LABORATORY COARSE AGGREGATE STUDIES

One very desirable goal in a skid-accident reduction program is a system for the prequalification of available aggregates with respect to the friction quality they can be expected to offer when incorporated in highway surfaces subjected to a variety of operating conditions. Under such a system, aggregate selection can be based specifically on need at the location of use.

In its overall skid-accident reduction program, Illinois has entered into two studies aimed at the establishment of procedures for rating aggregates based on their frictional properties. One of the studies is complete; the other is just beginning. Following is a discussion of each.

University of Illinois Coarse Aggregate Study

A preliminary study that searched for possible relationships between the results of a variety of laboratory tests applied to both aggregates and pavement samples and the results of friction tests on pavement surfaces

in the field was supported at the University of Illinois by the Illinois Department of Transportation during the period 1971-1975 (Illinois Project IHR-406). Full details are given in report (14). Because need for frictional improvement appeared greatest in the bituminous pavement area, work was concentrated in this area although consideration also was given to portland cement concrete pavements.

In this study, a number of test methods were used to determine the frictional and textural characteristics of 79 pavement surfaces located throughout the State. The testing was done by the standard ASTM E-274 skid trailer test method, the British Pendulum Tester method, and a stereo-photogrammetric method. Pavement cores were taken at the test sites for subsequent laboratory testing. Laboratory tests were conducted on aggregate and uncrushed stone from the sources that provided the aggregate for the pavements as well as on aggregate extracted from some of the bituminous pavement cores. Tests for acid-insoluble residue content, Na_2SO_4 soundness, Los Angeles abrasion resistance, hardness, accelerated wear characteristics, absorption capacity, bulk specific gravity, and mineralogical characteristics determined by optical microscopy and X-ray diffraction were included in the program.

Among the pavements tested were 26 PCC pavements, 28 Class I bituminous concrete pavements, and 15 Class A bituminous surface treatments. Sixty-six of the test sites were open-road locations, the remainder being at stop intersections. To insure a wide range of aggregates for study, test pavements were selected on the basis of coarse aggregate type used, geologic age of aggregate source, pavement class, and cumulative axle loadings. Aggregates initially were classified as gravel or crushed stone. The stone

later was subdivided into three groups; dolomite, limestone, and limestone-dolomite. The limestone-dolomite group consisted of limestones with a dolomite content in excess of 10 percent.

Locations of the pavements tested are shown in Figure 20; locations of their coarse aggregate sources are shown in Figure 21.

The search for possible relationships between measured pavement friction and aggregate properties was made principally by the application of linear correlation analyses to the test data. Most of the results were not unexpected. Many of the findings confirmed the results of previous research on the influence of general aggregate characteristics on measured friction.

The study confirmed the existence of a direct relationship between the ASTM E-274 test trailer friction number and both the British pendulum test skid number and the stereophoto test skid number. The relationships were found to be independent of pavement or aggregate type.

The British accelerated-wear apparatus was used to determine the aggregate "polished stone value." Use of the ratio of the polished stone value of the test aggregate divided by the polished stone value of a standard run in the same test cycle generally improved correlations with aggregate and pavement variables. The only correlation worth some consideration, and that not significant at the 0.95 confidence level, was between the temperature-adjusted test-trailer friction number and the polished stone value for Class I pavements.

A general direct trend was found between the temperature-adjusted trailer friction number and abrasion as measured with the Taber apparatus.

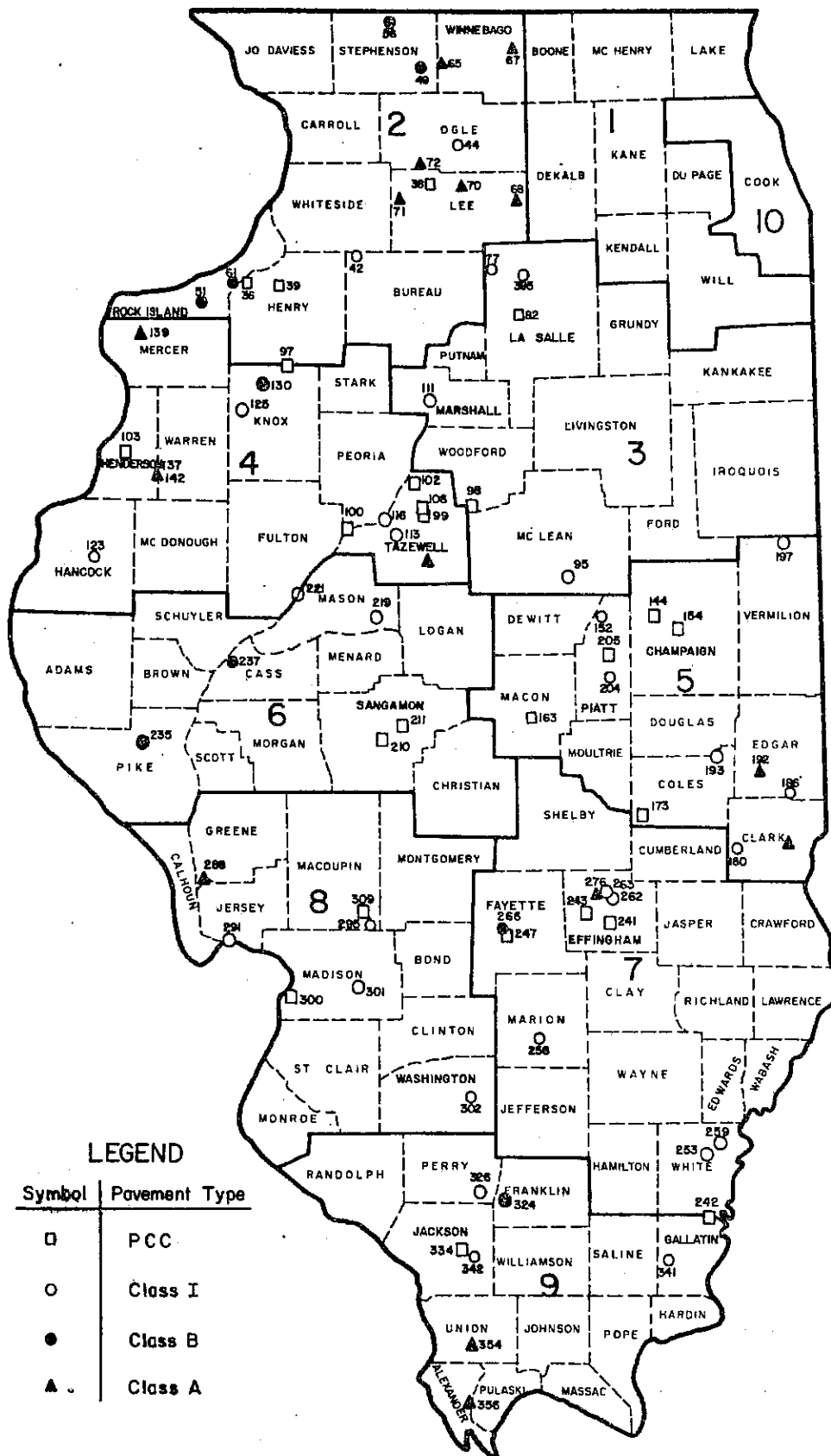


Figure 20. Location of Illinois Pavements Studied in the Present Investigation (14).

Figure 21. Location of Illinois Quarries which Supplied Coarse Aggregate for Pavements Shown in Figure 20 (14).

The correlation was strongest for dolomite, there being insufficient data for the other aggregate types.

A general relationship was seen to exist between the trailer friction number and aggregate from the Class I cores. A lower limiting value also appeared to exist between the trailer friction number and the plus No. 200 sieve acid-insoluble contents of extracted aggregates. This could serve to indicate a level at which the acid-insoluble content is the dominant aggregate feature controlling friction quality. A direct relationship also was seen to exist between the temperature-adjusted friction number and the mean crystal size of carbonate coarse aggregate as shown in Figure 22. The correlation was particularly significant with dolomite aggregate and appears to be the dominant factor influencing the aggregate studied. In addition, the mean temperature-adjusted trailer friction number had a direct relationship with the mean carbonate crystal size of the various carbonate aggregate groups. Dolomites had the largest and limestones the smallest mean crystal size.

Absorption capacity, percent acid insolubles, Los Angeles abrasion, and percent quartz content, although not showing highly significant relationships, showed general direct trends with the British polished stone value.

The amount of coarse aggregate in the surface also correlated with measured friction. This relationship was true for all carbonate aggregates tested, but particularly noticeable for limestone and limestone-dolomite aggregates.

The greatest difficulty in relating measured friction to individual aggregate properties stems from the variable condition of the pavement

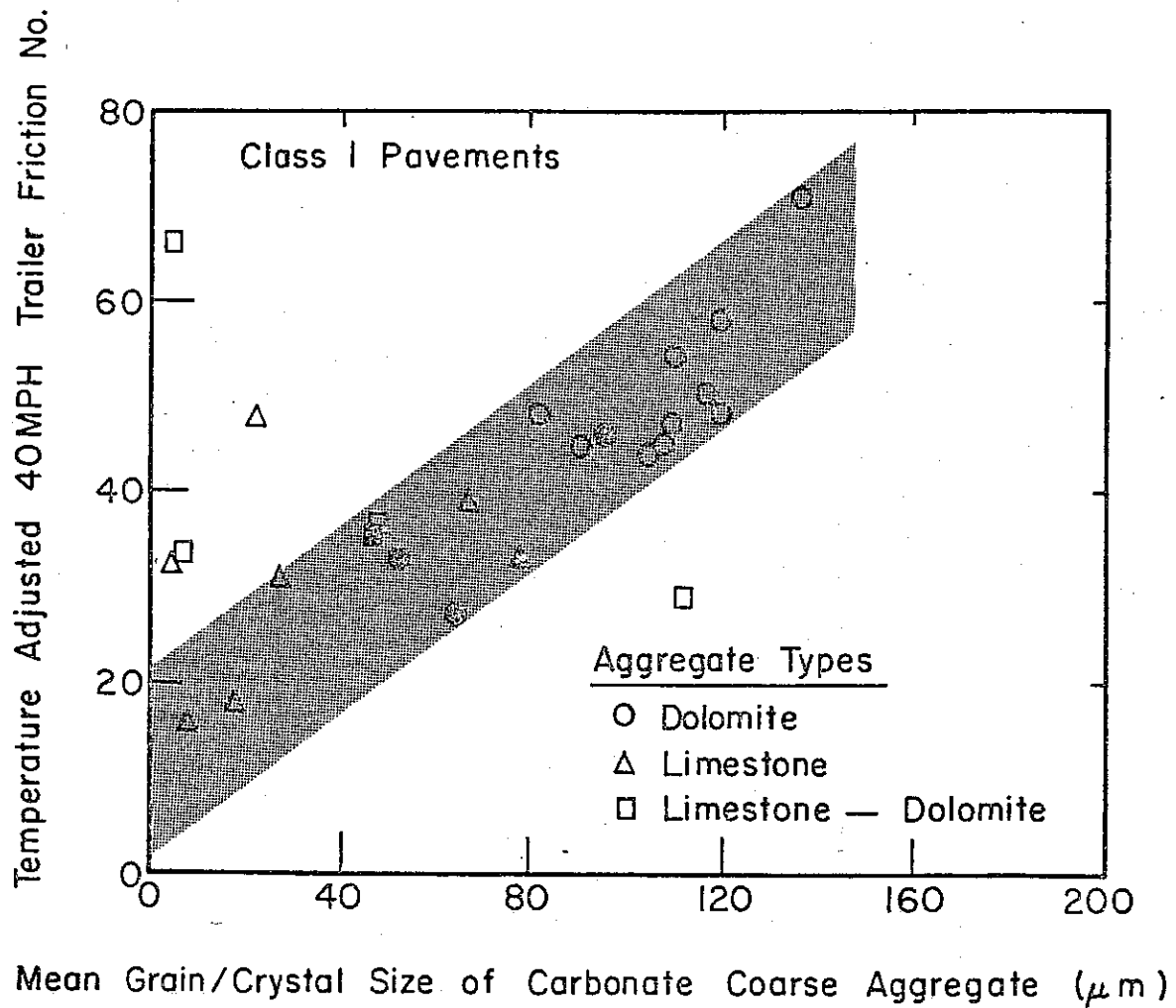


Figure 22. Temperature Adjusted Trailer Friction No. of Class I Pavements versus Mean Grain/Crystal Size of Carbonated Aggregate Determined by Petrographic Analysis of Pavement Cores (solid points are intersections and cross hatched area shows the trend of dolomite aggregates) (14).

itself. The present variability of pavement surfaces makes impossible the relating of aggregate properties to measured friction by simple correlations. All aggregate types, but particularly the limestone and limestone-dolomite, appeared to be influenced by the amount of coarse aggregate in the pavement surface. To achieve a correlation between aggregate properties and measured friction, the surface must be normalized to a constant coarse aggregate content.

The petrographic method of thin section analysis used in the study had its drawbacks in determining accurately the quantity of coarse aggregate in the pavement surface. The quantimet method is recommended for this analysis, with the aggregate parameters adjusted in accordance with the total area of aggregate present. As an alternative, a stepwise multiple regression analysis could be used with the total area of aggregate in the pavement surface used as an independent variable. This type of analysis should help in showing the influence of the individual aggregate characteristics within the rock groups on pavement friction quality. In this study, the Class I pavements with dolomite coarse aggregate had reasonably similar surface compositions and texture. So, the properties of the dolomite aggregate became the dominant influence in controlling friction rather than the properties of the pavement, such as quantity of aggregate present in the surface.

Aggregates within the same rock group will have different characteristics which can be the dominating influence on friction quality. For example, friction may be controlled by the acid-insoluble content of a dolomite aggregate when the insoluble content is over 25 percent, but by the crystal size of another dolomite aggregate with an acid-insoluble content of less than 10

percent. In aggregates with acid-insoluble contents between 10 and 25 percent, both factors might contribute to friction quality without one being dominant over the other. To determine these types of relationships, an adequate laboratory test must be developed for evaluating the friction quality of individual aggregates. Present tests are adequate only for making generalizations about rock groups. This may be partly a problem of inadequate normalization of the measured friction of pavements that renders direct correlations with laboratory tests impracticable.

In summary, the data available for study were sufficient only to show certain trends between the friction quality of pavement surfaces and the characteristics of Illinois coarse aggregates, and insufficient for recommending specifications for the friction quality of aggregates. However, the study did provide a base for suggestions on features on which future investigations might well concentrate. They are:

- (1) Coarse aggregates in pavements that have attained "stability," such as Class I bituminous concrete pavements with more than 1.5×10^6 axle applications.
- (2) The broadest possible range of carbonate coarse aggregates used in Illinois, preferably through extraction from pavement cores.
- (3) Normalization of the pavement surface (especially the area occupied by coarse aggregate) prior to attempting direct correlations with aggregate characteristics.
- (4) Development of adequate laboratory tests to evaluate friction quality.
- (5) Development of specific tests for various coarse aggregate groups rather than a "universal" test for all aggregates.

A broader selection of carbonate aggregates will permit determination of the validity of lower limiting values for the relationship of friction quality to variables such as acid-insoluble content, abrasion hardness, Los Angeles wear, etc. The establishment of these lower limiting values can assist in setting specifications. The broader selection also will permit a more detailed study of such relationships as that between mean crystal size and friction quality. Data from the completed study could provide the basis for a tentative specification relating crystal size of dolomite aggregate to friction quality, but more data would be needed to confirm the results.

The use of aggregate from pavement cores will provide information on the exact aggregate used in the pavement. This should overcome the problem of attempting correlations of friction quality with the results of tests on aggregates that can differ appreciably from those incorporated in the pavements.

Normalization of the pavement parameters, particularly the proportion of the total surface area occupied by the coarse aggregate, will allow a correlation of friction quality with the relative value of any given aggregate characteristic. As an alternative, the proportion of the total area occupied by the coarse aggregate can be considered as an independent variable in a multiple linear regression analysis.

Reliable test methods must be developed to evaluate the friction-quality potential of aggregate. However, because different aggregate characteristics can have a dominating effect on friction quality, depending on relative content and perhaps other factors, and also because they vary depending on general aggregate groupings (dolomite, limestone, igneous, etc.), a search

for tests to evaluate aggregates within each aggregate group ultimately may prove to be more fruitful than a search for a "universal" evaluation test.

North Carolina Wear and Polishing Machine Evaluation

A small-wheel circular track for laboratory evaluation of the wear and polishing of aggregates and pavement surfaces patterned after the North Carolina State University Wear and Polishing Machine - Mark II is being fabricated by the Bureau of Materials and Physical Research in Illinois. The Mark II is a second-generation device developed to rate aggregates and paving mixtures as to their skid resistance after exposure to wearing and polishing. It is a small-diameter circular track machine capable of accelerated wearing and polishing of pavement specimens. Correlation and comparison are accomplished by the use of control specimens that are tested simultaneously with regular test specimens. Track capacity is twelve individual specimens per test run.

The actual wearing and polishing is provided by the action of four smooth pneumatic tires to eliminate as nearly as possible the variables associated with tread wear. The wheels can be adjusted for camber and toe-in and toe-out to provide scrubbing action for polishing without the aid of water or grinding compounds.

Upon completion of the device, a program for developing correlations between machine test results and the friction characteristics of pavements in service in Illinois will be established in an effort to make the machine useful in rating the friction quality of new aggregates and surfaces before extensive field use.

IN THE FUTURE

Bituminous Pavements

A monitoring and evaluation of the overall performance of dense-graded bituminous concrete surfaces constructed under the Class I, Mixtures C, D, and E specifications, open-graded asphalt friction courses, the more promising sand-asphalt mixtures, and the new sprinkle treatment will continue. Sections of new pavement will be added to the studies as necessary to provide adequate representation. All principal types of coarse aggregates in current use will be covered and a variety of traffic conditions will be represented. Other variables also are under consideration for inclusion. Friction numbers will be determined with the ASTM E-274 test trailers. Most of the friction testing will be done at the standard test speed of 40 mph. However, testing also will be done at other speeds in selected situations to obtain friction number-speed gradients. Some testing also may be done with both the standard treaded tire and a smooth tire to see whether meaningful relationships can be developed. Friction data will be obtained throughout the useful lives of surfaces in selected instances. Cumulative traffic will be considered in performance analyses. Evaluations will include examinations and analyses of accident data.

The search will continue for alternate aggregates or aggregate combinations that will provide stable, durable, and skid-resistant bituminous surfaces. More economical alternatives to existing high-cost aggregate are of particular interest, although at this time none have been identified for evaluation in the immediate future.

Portland Cement Concrete Pavements

Monitoring and evaluation of the performance of representative portland cement concrete pavement surfaces textured in accordance with the currently specified artificial turf/transverse tining process will continue in a manner similar to that outlined for bituminous surfaces in the immediately preceding section.

Experimental effort to improve the friction quality of portland cement concrete pavements through better texturing procedures will continue. Currently, a small trial installation following existing specifications, except for a variable spacing of tines repeated at 6-in. intervals, is planned for inclusion in a planned construction project. A control section textured under current specifications will be installed to serve under the same traffic conditions. Friction tests and noise-level tests will be made immediately following construction and as often thereafter as seems appropriate. Other variables influencing the friction quality of portland cement concrete pavements will be studied as the need arises.

Laboratory Coarse Aggregate Study

When the small-wheel circular track for laboratory evaluation of wear and polishing of aggregates and pavement surfaces, as mentioned in an immediately preceding section of the report, has been completed, a wide-ranging program for the development of correlations between machine test results and the frictional performance of pavements in service in Illinois will begin. Hopefully, the machine will be useful in prequalifying new aggregates and surfaces for service in Illinois before construction.

CHAPTER FOUR

IDENTIFICATION AND CORRECTION OF HAZARDOUS SKID-ACCIDENT LOCATIONS

A systematized program for identifying and correcting hazardous skid-accident locations is one of the principal elements of the continuing comprehensive highway safety improvement program conducted in Illinois in compliance with Federal law and Federal Highway Administration directives. Activity in the skid-accident-reduction element of the program is described, along with activities in the many other elements, in a report compiled annually by the State of Illinois, Department of Transportation, Division of Highways, Bureau of Traffic, entitled, "Evaluation and Report of the Highway Safety Construction Program." The most recent issue of the report is that of August 1980 for Fiscal Year 1980 (12). The report is the product of information from several offices in the Illinois Department of Transportation. The Division of Traffic Safety is the source of the accident information that serves as a base in developing and evaluating the various safety programs. Other sources of information include the Highway Safety Construction Committee and the Bureaus of Local Roads and Streets, Location and Environment, Design, Materials and Physical Research, and the Program Management and Planning Service Sections.

ACCIDENT DATA BASE

Illinois has a computerized Accident Information System for matching and coding accident reports implemented in its present form in 1973. The majority of the cases processed include a report from each driver

involved in an accident and a police report. A special and more detailed report is prepared by District personnel for accidents on the State system involving two or more fatalities. After location coding and map spotting, case files are microfilmed and put through a statistical coding process. Data from the statistically coded cases are put into the computer and become the data base for summary and report preparation. The data have a variety of uses, such as the preparation of high-accident rate maps, evaluation of design and construction policies, and analysis of high-accident locations and alleged deficiencies in design, construction, and maintenance.

IDENTIFICATION OF HIGH-ACCIDENT LOCATIONS

The Illinois Highway Safety Improvement Program is begun each year with the preparation by the Division of Traffic Safety of an "Accident Rate Map" which indicates the locations along a highway that have accident rates above average. For sections of highways, accident rates are expressed as the number of accidents per 100 million vehicle-miles (HMVM) of travel. For intersections, accident rates are expressed as the number of accidents per million vehicles (MV) passing through the intersection.

For preparation of the Accident Rate Map, a spot map and a traffic volume map are required. A spot map is a county or city map maintained by the Division of Traffic Safety showing, by spots, the locations of accidents that have occurred during the year. The traffic volume map shows the average daily traffic volumes computed for each highway.

All high-accident locations are selected by a quality control method that provides assurance that the high accident rates are not

likely to be due to chance alone. The following formula is used for this purpose:

$$R_c = R_a + K \frac{R_a}{M} - \frac{1}{2M}, \text{ where}$$

R_c = Critical accident rate, in accidents per HMVM for sections, and in accidents per MV for spots.

R_a = Average accident rate on a specific category of highways (rural nonfreeway, Chicago expressways, etc.) per HMVM for sections, and on a specific category of spot locations (rural intersections, urban intersections, etc.) per MV for spots.

M = HMVM of exposure for the study period at location for sections and MV passing through spot locations.

K = A constant, the value of which determines the level of probability

The average accident rates are calculated yearly for the various categories that have been established. Intersection accidents are excluded from the calculation of accident rates for sections. Three values of K are used in the formula to help set priorities: 1.5, 3.0, and 4.5. The greater the value of K , the greater is the possibility that the accidents did not occur by chance alone. Color-codings are used on the Accident Rate Map to distinguish between the different values of K . Nomographs are used to speed computations.

A computer plotting program developed in FY 75 is used to provide the District Offices with collision diagrams for all high-accident-rate intersections. Collision diagram printouts are prepared for all other locations. Individual accident reports at a location under investigation are made available on request.

In the improvement selection process, priority is given to shorter sections with higher values of K as compared with continuous longer sections with lower values of K.

The Division of Traffic Safety identifies approximately 2,000 high-accident locations of all types each year, well above the relatively low number of improvements that can be made each year with available funding.

Among the listings of high-accident locations provided by the Division of Traffic Safety to the Bureau of Traffic each year is a listing of locations on State-maintained highways where skidding appears to have been a contributing factor. Currently, to be considered for improvement, the skid locations must meet all of the following criteria:

- (1) Approximately 60 percent of the total accidents are of a type that could involve skidding (rear-end accidents, running off the roadway on dry pavement, and all accidents on wet pavements).
- (2) Approximately one-third of the total accidents have occurred on wet pavements (accident on snow or ice-covered surfaces excluded).

(3) There is a total of at least three accidents at the location. Changes in the criteria to place more emphasis on wet-weather accidents are now being considered.

During the past seven years from FY 73 through FY 79, skid-resistance improvements have been made at more than 160 sites of low skid resistance. Friction tests with the ASTM E-274 skid trailer were made at nearly 1,200 potential locations in selecting these sites for improvement.

A total of 161 high-accident locations on State highways (94 sections and 57 spot locations) where skidding appeared to be a contributing

factor were identified in FY 80. An additional 10 possible skid locations were identified on the Interstate system. Following a review of the listing of locations by the Districts, and a deletion of those scheduled for structural improvement and the addition of those of special concern in the Districts, a total of 143 were friction tested with the E-274 trailer. Available funds limited the scheduling of friction-quality improvements to 18 locations during FY 80.

ESTABLISHMENT OF ANNUAL SAFETY IMPROVEMENT PROGRAM

When the District Offices each year receive the maps identifying high-accident sections and spots in their jurisdictions, they are used with accident collision printouts and the computer-generated collision diagram plots to analyze the high-accident locations. Field investigations are made at many locations to observe operational problems and to make video tapes if appropriate. Friction numbers determined with the ASTM E-274 skid trailer for sections that appear eligible for frictional improvement are considered where this type of improvement is contemplated.

After the Districts have listed potential safety improvement projects in priority order, the lists are submitted to the Central Office. Here they are reviewed by the Highway Safety Construction Committee comprised of a representative of the District Highway Offices, the Division of Traffic Safety, the Bureaus of Location and Environment and of Traffic, and the Program Management Section. After committee review, each District's listing is reviewed jointly with a District representative. Following is a listing of major items given consideration by the Highway Safety Construction Committee in establishing the annual Hazard Elimination Program.

- (1) High-Accident Rate Location - Project location must appear as a spot or short section on the current rate map, except for skid-proofing projects proposed on the basis of low skid numbers (average SN less than 30).
- (2) Total Cost of Project - If excessive, little further consideration is given. The maximum is generally \$500,000, with preference given to projects in the \$50,000 to \$250,000 range.
- (3) Total Number of Accidents - A site may qualify as a high-accident rate location but have too few accidents to establish any pattern or be cost-effective.
- (4) Accident Patterns - The overrepresentation of certain types of accidents (i.e., above average percent of wet-weather, night-time, rear-end, angle, etc.) increases the probability that a related improvement can correct the problem.
- (5) Applicability of Proposed Treatment - Does the proposed treatment relate to the accident problem? Will it reduce the number and severity of accidents? Would a less costly improvement achieve the same results?
- (6) District's Recommended Priority - Special consideration is given to District's high priority projects.
- (7) Benefit/Cost Ratio - The ratio should be favorable. The most meaningful use of the benefit/cost ratio at this time is comparing similar type and size projects.
- (8) Accident Trends - When comparing similar projects, an increasing accident trend is given precedence over a decreasing one. Current year accidents (which Districts may not have available) may be used to verify accident trends of marginal projects.

- (9) Compatibility with Scheduled Improvements - Would the proposed improvement be unneeded or require reconstruction when future scheduled improvements are initiated? Will the accident patterns or traffic volume change?

The following method has been developed for determining the relative benefit-to-cost ratios for proposed highway safety construction projects. This information is utilized in conjunction with the other Safety Committee considerations in the selection of cost-effective projects.

Two basic assumptions are made for the method:

1. Only certain types of accidents are affected by specific types of improvement.
2. Accident types that can be affected by the proposed improvement will be reduced to the proportion that those types are of the total accidents reported on State-marked and maintained highways.

Assumption 2 should cause the outcome of the Benefit/Cost calculation to be conservative since only a reduction to the Statewide average is assumed.

To perform the analyses, the following tables were developed. They are not included in this report in the interest of brevity, but are included in (12).

- (1) A matrix showing the types of accidents that are affected by type of improvement.
- (2) 1977 Statewide Percentage of Accidents by Type of Collision versus class of Trafficway.
- (3) Statewide average accident rates by highway system.

- (4) Expected service life of various spot safety improvements.
- (5) Weighted costs of accidents by highway system.
- (6) Location, improvement, type, accident data for Safety Improvement Projects by District.
- (7) Projected Benefit/Cost ratio for Safety Improvement Projects by District.

The final two tables listed above, filled out for illustration in the example to follow, are those prepared for use by the Highway Safety Construction Committee and are shown in Figures 23 and 24. The table in Figure 23 shows the District's priority for the project location, improvement type, estimated cost, service life and various accident data for the Committee's use as well as for use in preparation of the Benefit/Cost ratio. The table in Figure 24 shows the step-by-step method utilized in arriving at the projected benefit/cost ratio for each District's proposal.

To illustrate the method, the priority 1 project for District 8 is used as an example and completed tables are used in the illustration. Note that each column on the table of Figure 24 has been numbered for ease of reference in explaining the method. Several of the columns are self-explanatory, but for uniformity each is explained below:

Column 1 - Location number or the District's priority number.

In the example this is priority number 1 for District 8.

Column 2 - Estimated cost in dollars. This is the District's estimated cost of the project of \$72,000.

Column 3 - Estimated life in years. This comes from the service life table and in the example is 15 years.

P R I O R I T Y	D I S T R I C T # 8	L O C A T I O N	I M P R O V E M E N T	E S T. C O S T (\$)	S E R V I C E Y R S.	(TA) T O T A L A C C I D E N T S			(AR) A C C I D E N T R A T E			(AA) A F F E C T E D A C C I D E N T S (%)			'77 S T R A T T E E (%)	B E N E F I T (B/C)
						'75	'76	'77	'75	'76	'77	'75	'76	'77		
						YRS.										
1	ILL. 159 at Douglas Road-St. Clair Co. (Spot)	Signal Installation	72,000	15	8	7	11	3.5 (Y)	3.0 (Y)	4.6 (G)	100	96.3	10.08			
2	ILL. 111 at Bus. 40 in Fairmount City-St. Clair Co. (Spot)	Geometrics and Signal Modernization	150,000	15	28	31	20	5.9 (R)	6.5 (R)	3.6 (Y)	90.0	97.8	2.15			
3	ILL. 143 at 9th Street in Wood River-Madison Co. (Spot)	Signal Modernization	60,000	10	9	13	7	1.8 (NC)	2.6 (NC)	1.3 (NC)	71.4	97.8	0.0			
4	ILL. 143 from 0.1 mi. East to 0.1 mi. West K-Mart Entrance in Wood River-Madison Co. (0.20 mi. Section)	Left Turn Lanes	115,000	15	-	2	14	(NC)	274 (NC)	1728 (R)	71.4	71.7	18.26			
5	ILL. 111 at ILL. 162-Madison Co. (0.20 mi. Section)	Left Turn Lanes Signal Installation	235,000	15	13	19	21	1.8 (NC)	2.6 (Y)	3.3 (Y)	100	97.8	1.83			
6	ILL. 15 at St. Libory Curve-St. Clair Co. (Spot)	Relocate Curve and Bridge	360,000	15	3	7	3	1.5 (Y)	3.5 (R)	1.5 (G)	100	59.6	0.25			

Figure 23. Example of District Tabulation of Data Pertinent to Safety Improvement Project Recommendations (12).

(1) LOCATION NUMBER	(2) ESTIMATED COST (S)	(3) ESTIMATED LIFE (YR.) ⁽¹⁾	(4) STATEWIDE RATE (RA)	(5) TOTAL ACCIDENTS (TA)	(6) AVERAGE DAILY TRAFFIC (ADT)	(7) LENGTH (MI.)	(8) LOCATION ACCIDENT RATE	(9) ACTUAL AFFECTED ACCIDENTS (%)	(10) AFFECTED ACC. RATE $(8) \times (9)$	(11) UNAFFECTED RATE $(8) - (10)$	(12) STATEWIDE AVERAGE DISTRIBUTION OF AFFECTED ACC. (%)	(13) ANTICIPATED AFF. RATE $(4) \times (12)$	(14) SAVINGS - AFFECTED RATE $(10) - (13)$	(15) SAVINGS - AFFECTED ACCIDENTS $(14) \times (6) / 2740$	(16) BENEFITS (S) $(15) \times \text{ACC. COST}$	(17) BENEFIT/COST ANNUAL $(16) / (2 \div 3)$
1	72,000	15	1.55	11	6,600	-	4.6	100	4.60	0	96.3	1.49	3.11	8	48,400	10.08
2	150,000	15	2.10	20	15,150	-	3.6	90.0	3.24	0.36	97.8	2.05	1.19	6	21,540	2.15
3	60,000	10	2.10	7	14,300	-	1.3	71.4	0.93	0.37	97.8	2.05	-1.12	0	0	0.0
4	115,000	15	3.90	14	11,100	0.20	1728	71.4	1234	494	71.7	280	954	39	140,010	18.26
5	235,000	15	2.10	21	17,400	-	3.3	100	3.3	0	97.8	2.05	1.25	8	28,720	1.83
6	360,000	15	0.25	3	5,525	-	1.5	100	1.5	0	59.6	0.15	0.35	1	6,050	0.25

Figure 24. Example of District Tabulation of Data for Benefit/Cost Analysis Relating to Proposed Safety Improvement Projects (12).

Column 4 - Statewide average accident rate for the particular facility. This is taken from the statewide average accident rate table. In this case, the location is a downstate rural intersection or spot having an average rate of 1.55 accidents per million vehicles entering the intersection.

Column 5 - Total accidents reported as occurring at the proposed location in 1977. In this case, according to the table of Figure 23, there were 11 accidents at the location.

Column 6 - Average Daily Traffic (ADT). Taken from the ADT maps produced by the Bureau of Planning. In this example, the ADT was 6,600 for 1977.

Column 7 - Section length in miles. This is utilized along with number of accidents and ADT to calculate the accident rate for a section. Since the example chosen is not a section, the column is left blank.

Column 8 - Location accident rate. This is calculated different ways for sections and intersections. For sections the equation is $\text{Accident Rate} = \frac{\text{Number of Accidents} \times 10^8}{\text{ADT} \times \text{Length} \times 365}$

This yields the accidents per 100 million vehicle miles travelled over a section.

For an intersection the equation is $\text{Accident Rate} = \frac{\text{Number of Accidents} \times 10^6}{\text{ADT} \times 365}$

For the intersection example, this yields an accident rate of 4.6 per million vehicles entering the intersection.

Column 9 - Actual affected accidents (percent). This is a percentage calculated by first obtaining the accident types that could be affected by the improvement from the appropriate table. For signal installation projects the accident types affected are: pedestrian, fixed-object, rear-end, sideswipe same direction, angle, and left- and right-turning accidents. Furthermore, the accident types are likely to be affected during all light and pavement conditions.

The second step in calculating the percent of actual affected accidents is to check the collision diagram information printout for number of accidents of these types occurring and divide by the total number of accidents. In the example, all accidents are of the type that could be affected and the figure calculated is therefore 100 percent.

Column 10 - Affected accident rate. Calculated by multiplying Column 8 by Column 9, this result gives that portion of the location's accident rate which could be affected by the improvement. For this location the value is $4.60 \times 100 \text{ percent} = 4.60$.

Column 11 - Unaffected rate. Obtained by subtracting the affected rate from the total accident rate at the location (Column 8 - Column 10). The results give that portion of the accident rate which probably would not be affected by the improvement. For the location the value is $4.60 - 4.60 = 0$.

NOTE: Considering that one of the assumptions made was that affected accident types would be reduced to the proportion that those types are of the total accidents occurring on State-marked and maintained highways, the following two steps are needed.

Column 12 - Statewide average distribution of affected accidents (percent). This value is obtained from the table that shows average accidents by highway system according to geographical location, i.e., rural, urban, Chicago. Further, categorizations are made of intersection signalized, intersection unsignalized, and non-intersection accidents for each type of geographical locations. The types of accidents affected were previously determined. Going into the table showing average accidents by highway system, it is found that at Rural Signalized Intersections the percentages of affected type accidents on a Statewide bases are:

<u>Collision Type</u>	<u>Percent</u>
Pedestrians	0.3
Fixed-Object	8.6
Rear-End Both Moving	3.3
Rear-End One Stopped	29.5
Sideswipe Same Direction	5.0
Angle	11.4
Turning	<u>38.2</u>
Total Affected Type	96.3

Therefore, 96.3 percent of accidents occurring at rural signalized intersections are of the types that could be affected.

Column 13 - Anticipated affected rate. This value is obtained by multiplying Column 4 (Statewide rate for rural intersections) by Column 12 (Statewide average distribution of affected accidents).

It is to this value that the rate of affected accidents is expected to drop. For the example the result is $1.55 \times .963 = 1.49$ anticipated rate of affected accidents following the improvement.

Column 14 - Savings in affected rate. This value is obtained by subtracting Column 13 (anticipated rate of affected accidents) from Column 10 (actual current rate of affected accidents). Thus $4.60 - 1.49 = 3.11$, or, the reduction in the affected rate to be brought about by the improvement is 3.11 accidents per million vehicles entering the intersection.

Column 15 - Number of accidents eliminated. Having obtained the value for the anticipated reduction in the affected rate, it is now a matter of working backward through the accident rate equation to obtain a number of accidents to be reduced. In the case of an intersection the equation is $\text{Accident Rate} = \frac{\text{Number of Accidents} \times 10^6}{\text{ADT} \times 365}$

Solving this equation for a rate of 3.11 and an ADT of 6,600 we obtain $\text{Number of Accidents Reduced} = \frac{10^6}{3.11 \times 6,600 \times 365} = 7.5$

Therefore, approximately eight accidents will be eliminated by the improvement.

Column 16 - Benefits. This value is obtained by multiplying Column 15 (the number of accidents prevented) by the weighted average cost of a rural accident taken from the appropriate table of weighted accident costs. For rural accidents the average weighted cost per accident has been determined to be \$6,050. Therefore, the benefits are:

$$8 \text{ Accidents reduced} \times \$6,050/\text{accidents} = \$48,400$$

Column 17 - Annual Benefit-to-Cost ratio. The ratio is obtained by dividing Column 16 (average annual benefits) by the average annual cost of the project. This average annual cost is determined by dividing Column 2 (estimated cost of the project) by Column 3 (estimated service life of the project). For the example, the final benefit to cost ratio calculated is

$$B/C = \$48,400 / (72,000 / 15) = 10.08$$

It should be emphasized that the present procedure for projecting project benefit is an interim measure and will be modified once we have amassed sufficient information to more accurately project what kinds of actual reduction can be expected from a specific improvement.

After the proposed programs from all Districts have been screened, those that appear to be the most effective are selected for inclusion in the annual safety improvement program.

Proposals for local improvements are solicited by and submitted through the District Offices to the Central Bureau of Local Roads and Streets. At this point they are screened and transmitted to the Safety Construction Committee where they compete with State projects for available funding.

After the Highway Safety Construction Committee has established priorities and selected the safety improvements to be included in the current fiscal year's program, the District Offices are notified of the selected improvements on the State-maintained highway system and the Bureau of Local Roads and Streets of the improvements on local highways, and the process of development into construction projects and the letting

of construction contracts is undertaken. Generally, none of the selected projects is given scheduling priority. If a project does not get implemented in the fiscal year scheduled, to receive further consideration it must be resubmitted to the Safety Construction Committee and compete with the new projects proposed for that fiscal year. Any substitutions in the approved program or substantial changes in project scope or estimated cost must receive the Safety Construction Committee's approval.

PROGRAM EVALUATION

Evaluation of the effectiveness of frictional improvements in reducing the number and severity of accidents and potential accidents in Illinois has been centered on frictional improvement projects that have been built under the Hazard Elimination Safety Program of the Illinois Safety Construction Program. Frictional improvement is one of several types of improvements included in the program.

The analysis of completed safety projects involves the use of a simple benefit/cost ratio. This ratio is determined through the application of National Safety Council accident cost statistics, Illinois Highway Safety Improvement Program accident statistics, and Illinois and National Cooperative Highway Research Program estimates of project service lives. Each accident is assigned a dollar value based on severity using National Safety Council cost per death, personal injury, and property damage accident. These values are multiplied by the difference in before-and-after accident numbers and then by the estimated project life. This number is then divided by the total project cost, yielding the benefit/ cost ratio.

At the present time, a preliminary assessment of the benefit/cost ratio is made based on two years of "before" traffic data and one year of "after" data. A final assessment of the benefit/cost ratio is made based on two years of "before" and two years of "after" data. Final-assessment results for 85 projects mostly from the FY 77 program (and completed by January 1, 1978) are shown in Table 18.

The benefit/cost ratios shown in Table 18 are based on 1978 National Safety Council estimates of accident costs. The accident figures shown are total accidents for the location rather than accidents likely to be affected by the improvement. Greater precision is not believed to be necessary at this stage of analysis because of the minimal number of locations included in the analyses. It is anticipated that, as additional data become available, the analysis can be expanded to contain an evaluation of accident histories that eventually will permit firm conclusions regarding relationships between various types of improvements and accident reduction (in number and severity). The analysis will continue with this goal in mind.

It will be noted in the table that 14 frictional-improvement projects (all friction-improvement overlays) showed a respectable average benefit/cost ratio of 1.90 in the two-year "before" and "after" study using projects mostly from the FY 77 program. It is of interest that the previous year's analysis of 19 friction-improvement overlay projects mostly from the FY 76 program showed an unfavorable benefit/cost ratio of -1.7. Without the inclusion of three consecutive intersections that showed significant increases in accidents following improvement, a plus benefit/ cost ratio would have resulted. The relatively wide variation is the average

TABLE 18

TWO YEARS BEFORE/TWO AFTER BENEFIT/COST ANALYSIS (12)

(Sec 209 Projects)

Improvement Type	Number of Projects	Before F After F	Before PI After PI	Before PD After PD	Benefits/Costs	Estimated Service Life (Years)
Channelization	2	0/0	81/62	132/92	3.83	20
Channelization & Traffic Signals	15	1/0	601/419	912/635	5.44	17.5
Traffic Signals	23	1/1	272/338	615/580	-7.06	15
Sight Distance						
General Intersection Improvement	12	0/0	201/112	280/206	4.58	15
Widening	2	1/0	54/66	39/47	7.02	10
General Cross Section	6	1/0	91/61	196/167	10.27	12
Grooving						
Friction Improvement Overlays	14	1/2	182/111	233/135	1.90	5
Horizontal Alignment	1	2/0	10/0	4/2	31.59	15
Roadside Improvements	5	0/1	62/85	85/108	-21.31	15
Widening Bridge	1	0/0	46/3	16/5	18.14	15
Guardrail	2	1/0	6/1	2/18	42.63	10
Other	2	0/0	160/113	319/262	509.60	10
Total	85					

Legend: F = Fatal, PI = Personal Injury, PD = Property Damage

Note: Mostly projects in FY 77 program completed before January 1, 1978

benefit/cost ratios based on the two sets of data and the noted relatively strong influence that a few projects can have on the average ratio support the belief that considerably more data are needed to draw firm conclusions regarding the advantages of the improvement.

No effort has been made at this stage to apply the benefit/cost analysis to different types of overlay because of the small amount of data available.

An application of the benefit/cost analysis to seven grooving projects in the FY 76 program showed an average ratio of -1.7. Here again, the computed average is based on a minimal amount of data.

A small number of other evaluations of accident data have been made in connection with friction improvement as described elsewhere in the report, and more are contemplated as the need arises and as the amount of accumulated data reaches a size that will produce more reliable results.

IN THE FUTURE

Information available at this time suggests that, within the foreseeable future, the Illinois program for identifying and correcting hazardous skid-accident locations will continue to follow the general format that is now established. The scope of the program will reflect available funding. Improvements that develop from studies now in progress for identifying individual accident sites more accurately, for relating accidents more precisely to probable cause, and for better identification of hazardous locations, will produce improvements in identifying locations with a high potential for skidding accidents. Improvements under development in pavement design, construction, and

maintenance in specific relation to friction quality will provide better correctional techniques. Planned efforts to improve and expand accident data use and benefit/cost analyses should produce improvements in performance evaluations.

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U.S. DEPARTMENT OF TRANSPORTATION

FEDERAL HIGHWAY ADMINISTRATION

SUBJECT SKID ACCIDENT REDUCTION PROGRAM	FHWA TECHNICAL ADVISORY T 5040.17 December 23, 1980
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- Par. 1. Purpose
 2. Background
 3. Skid Accident Reduction Program
 4. Pavement Design, Construction, and Maintenance
 5. Wet Weather Accident Location Studies
 6. Pavement Skid Resistance Testing Program

1. PURPOSE. To provide guidance for State and local highway agencies in conducting skid accident reduction programs.

2. BACKGROUND

- a. This Technical Advisory provides a general overview of factors that should be considered as elements of any Skid Accident Reduction Program. This Technical Advisory supports current Federal Highway Administration (FHWA) policy and will be revised as appropriate to reflect changes in policy as they occur.
- b. The existing requirements for skid resistance pavements are contained in several documents including Highway Safety Program Standard No. 12, Highway Design Construction and Maintenance (23 CFR 1204.4), Federal Highway Program Manual (FHPM) 6-2-4-7, Skid Measurement Guidelines for the Skid Accident Reduction Program. Other sources of technical advice are cited in the appropriate sections of this Technical Advisory.
- c. Highway Safety Program Standard 12 (HSPS No. 12) states that every State shall have a program of highway design, construction, and maintenance to improve highway safety. This program shall provide that "there are standards for pavement design and construction with specific provisions for high skid resistance qualities." The HSPS No. 12 also requires that each State have a "program for resurfacing or other surface treatment with emphasis on correction of locations or sections of streets and highways with low skid resistance and high or potentially high accident rates susceptible to reduction by providing improved surfaces." In discharging the responsibilities of FHWA, the Division Administrator

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should determine the acceptability of specification requirements and construction practices for placing, consolidating, and finishing both asphalt concrete and portland cement concrete pavements. Such determinations will rely on the highway agency to research, evaluate, and document the performance of the various aggregates, mix designs, and construction practices used.

- d. Even though the use of studded tires is beyond the control of most highway agencies, their use can cause significant wear on the pavement surface texture. For example, grooves sawed in concrete pavements have worn completely down in as short a time as 2 years. States are encouraged to ban or restrict the use of studded tires.
 - e. Legislative actions in recent years support a general duty of any highway agency to "... maintain the roadway in a reasonably safe condition. This would involve, in essence inspection, anticipation of defects, and conformity with generally accepted standards and practices."* The practical result is that highway agencies should have an organized system to identify and correct hazardous locations in a cost-effective manner, as well as a comprehensive pavement management program to design, construct, and maintain highways in conformance with reasonable standards. Such a systematic process is the best way to execute the highway agency's duty to maintain a reasonably safe roadway.
3. SKID ACCIDENT REDUCTION PROGRAM. Each highway agency is encouraged to develop and manage a skid accident reduction program to reflect the individual needs and conditions within the State. The purpose of a skid accident reduction program is to minimize wet weather skidding accidents through: identifying and correcting sections of roadway with high or potentially high skid accident incidence; ensuring that new surfaces have adequate, durable skid resistance properties; and utilizing resources available for accident reduction in a cost-effective manner. A program comprised of at least the following three basic activities, if faithfully implemented, should enable the highway agency to comply with HSPS No. 12.

* Engineering and Government Liability, David C. Oliver, FHWA, an unpublished paper presented to the American Road and Transportation Builders Association Local Officials Meeting, St. Louis, Missouri, August 23, 1978.

- a. The evaluation of pavement design, construction, and maintenance practices through its pavement management program to ensure that only pavements with good skid resistance characteristics are used.
- b. The detection and correction of locations with a high incidence of wet weather accidents utilizing (1) the State and local accident record systems, and (2) countermeasures for locations with high wet weather incidences, to ensure that existing highways are maintained in a safe condition.
- c. The analysis of skid resistance characteristics of selected roadway sections to:
 - (1) ensure that the pavements being constructed are providing adequate skid resistance,
 - (2) develop an overview of the skid resistance properties of highway systems,
 - (3) provide up-to-date information for the pavement management process, and
 - (4) provide data for use in developing safety improvement projects and the implementation of cost-effective treatments at appropriate locations.

4. PAVEMENT DESIGN, CONSTRUCTION, AND MAINTENANCE

a. Pavement Design

- (1) Current pavement design practices should be evaluated to ensure that skid resistance properties are durable and suitable for the needs of traffic. Consideration of skid resistance levels, texture, aggregate availability, traffic volume, traffic speed, type of facility, rainfall, construction and maintenance practices, and accident experience are basic elements in such evaluations. Evaluations should document the compliance with the requirement for skid resistant surfaces and provide basic data for use in choosing corrective actions for locations with high wet weather accident rates.
- (2) One principal result of the evaluations is the development of a performance history for each particular pavement used by each highway agency. The performance of the existing pavement designs

should be monitored and new designs should be evaluated to ensure that only skid resistant pavement surfaces are used. Information should be gathered as to the durability of a mix and the loss of skid resistance under traffic.

- (3) The level of skid resistance needed for a particular roadway depends primarily on the traffic volume, traffic speed, type of facility, and climate with additional consideration warranted at special locations such as steep hills, curves, intersections, and other sites which experience high demands for pavement-tire friction. It is desirable to have one or more "skid resistant mixes" which have durable and higher than usual frictional properties for use in these special areas.
- (4) A pavement surface may provide adequate skid resistance at low speeds, yet be inadequate for high speed conditions. Pavement surfaces, therefore, should be designed on the basis of properties at expected operating speeds.
- (5) The American Association of State Highway and Transportation Officials (AASHTO) Guidelines for Skid Resistant Pavement Design, 1976, provide detailed information on the design of surfaces for both flexible and rigid pavements. The major considerations follow:

(a) Flexible Pavements

- 1 The skid resistance evaluation of bituminous pavements should include a determination that the aggregate used in the top layer of future pavements is capable of providing adequate skid resistance properties when incorporated in the particular mix and that the mix should be capable of providing sufficient stability to ensure the durability of the skid resistance.
- 2 A bituminous pavement surface should contain nonpolishing aggregates. It is essential for good skid resistance that a mix design be used which allows good exposure of the aggregates. This

requires that the pavement surface mixture be designed to provide as much coarse aggregate at the tire-pavement interface as possible.

- 3 The open graded asphalt friction course (OGAFC), with a large proportion of one size aggregate, provides excellent coarse texture and exposes a large area of coarse aggregate. Guidance for this mix can be obtained from FHWA Technical Advisory T 5040.13, Open-Graded Asphalt Friction Courses, January 11, 1980.

(b) Rigid Pavements

- 1 The evaluation of portland cement concrete (PCC) pavements should include a determination that the finishing procedures, mix design, and aggregates provide the initial texture and necessary surface durability to sustain adequate skid resistance.
- 2 In PCC pavements, the initial and early life skid resistance properties depend primarily on the fine aggregates for microtexture and on the finishing operation for macrotexture. Specifications for texturing concrete pavements should be carefully selected and enforced to ensure a macrotexture pattern appropriate to the type of facility.
- 3 Regardless of the finishing or texturing method used, adequate durable skid resistance characteristics cannot be attained unless the fine aggregate has suitable wear and polish resistance characteristics. Research by the Portland Cement Association indicates that the siliceous particle content of the fine aggregate should be greater than 25 percent.

- 4 If pavement evaluation studies indicate that the coarse aggregates will be exposed by the surface wear and have a significant effect on skid resistance of pavement, it too should have a suitable polish resistance characteristic.
- 5 Metal tines, preceded by burlap or another type of drag finish, are recommended as being the most practical and dependable method of providing texture in PCC surfaces. Additional guidance can be obtained from FHWA Technical Advisory T 5140.10, Texturing and Skid Resistance of Concrete Pavements and Bridge Decks, September 18, 1979.

b. Pavement Construction

- (1) Highway agencies are encouraged to adopt a policy of "prequalifying" aggregates to be used in surface courses. Prequalifying is a method by which aggregates can be classified according to their friction, texture, wear, and polish characteristics. Classifications should reflect performance related to traffic volume, operating speed, percent trucks, climate, geometric design, and other appropriate factors. Design procedures should be established to ensure that aggregates can be selected for each project which are suitable to the needs of traffic.
- (2) Prequalification may be accomplished by one of the following, or a combination of both:
 - (a) A systematic rating of all fixed sources of aggregates (e.g., a commercial quarry which obtains aggregate from the same location for many years). Ratings should be based on standardized laboratory tests such as the American Society for Testing and Materials (ASTM) D 3319, Recommended Practices for Accelerated Polishing of Aggregates Using the British Wheel, or ASTM D 3042 Test for Insoluble Residue in Carbonate Aggregates, combined with data obtained from skid resistance tests of pavements in service. Other tests may be added or substituted if shown to predict pavement performance.

- (b) An evaluation and in-service history of the geologic or petrographic types of aggregates commonly used. Thus, when a new aggregate source is proposed, it can be accepted with minimum testing if an in-service history has been established for that type of aggregate.
- (3) Based on prequalification of aggregates, construction plans and specifications should define the friction quality of aggregate which will be acceptable. The following steps should be followed to assure acceptability of the as-constructed pavement surface course:
 - (a) After the contractor has identified the particular aggregates and asphalt to be used on a project, it is recommended that a mix design be performed with the actual ingredients being used. Aggregates should be checked to determine if they are from prequalified sources or are an acceptable petrographic type.
 - (b) Macrotexture and void content are important considerations in asphalt mixes. Since asphalts are often blended from several sources of crude oil that vary in temperature-viscosity characteristics, the mixing temperature should be determined for each project after establishing the characteristics of the selected asphalt. Allowable tolerances for asphalt content, mixing temperatures, and gradation should be established for each asphalt mix.
 - (c) Job control of asphalt mixes should be designed to ensure that desired skid resistance properties are obtained. It should be recognized that small changes in aggregate gradation or asphalt content may significantly affect the macrotexture of finished surfaces.
- (4) The frictional properties of pavement surface types should be randomly tested within 6 months after opening to traffic to verify that the anticipated characteristics are present. Evaluation tests should involve direct measures such as the skid tester (ASTM E 274), or an acceptable alternative, but may use surrogate measures such as those which evaluate texture (for example, ASTM E 303, Standard Method for Measuring Surface Frictional Properties Using the

British Pendulum Tester; and sand patch tests as described in the American Concrete Paving Association Technical Bulletin No. 19, Guidelines for Texturing Portland Cement Concrete Highway Pavements, Measurement of Texture Depth by the Sand Patch Method).

- (5) In cases where the skid resistance properties of a pavement are found to be questionable or inadequate, appropriate warning signs should be placed immediately as an interim measure. A complete evaluation and any remedial action needed should be effected as soon as possible.

c. Pavement Maintenance. The same procedures and quality standards used in construction should be used in the maintenance operations.

5. WET WEATHER ACCIDENT LOCATION STUDIES. The purpose of this type of study is to identify locations with high incidence of wet weather accidents, determine corrective measures, and take appropriate actions in a timely and systematic manner. This activity should be conducted as part of the highway agency's safety improvement program and should make effective use of the agency's accident data file. Items to be considered for retrieval from the accident and traffic records are total accidents (rate), wet weather accidents (rate), and the wet/dry ratio.

a. Identification of Wet Weather Accident Sites

- (1) Accident records, which are developed in compliance with Highway Safety Program Standard No. 9, Identification and Surveillance of Accident Locations, should be searched at least annually to identify sites which have a high incidence of wet weather accidents. It is essential to have a standardized highway location reference system for correlating data from different sources. Accident rates at a site will be of greatest value if:
 - (a) the traffic volume is relatively high (i.e., approximately 1,500 vehicles per day or greater),
 - (b) the period of accident data is at least two years, and
 - (c) rainfall data are available for the same period as the accident data.

- (2) Rainfall patterns for the years in which skid resistance and accident data were compiled should be acquired for each area in the highway agency's jurisdiction. A suggested method is presented in Appendix A.
 - (3) There are several methods in use by highway agencies to evaluate wet weather accident locations. One such method is the Wet Safety Factor (WSF), which is presented in Appendix A.
- b. Field Review. A list of all sites ranked in order of WSF or another appropriate measure should be prepared as the basic list of candidate sites for remedial treatments. The selected locations should then be skid tested and reviewed by a team representing various disciplines such as highway materials, design, construction, maintenance, traffic and safety. See Appendix B for skid testing procedures. The review team should determine probable reasons for the high incidence of accidents and recommend corrective actions. Once the review team has recommended appropriate corrective treatments, a priority list of projects can be prepared based on benefits and expected costs.
- c. Priority Program. An assessment should be made of the benefits relative to the cost of providing remedial treatments for high priority projects. A number of highway agencies have their own methods for conducting benefit cost analyses of alternative remedial treatments. Some of these remedial methods are tied into traffic engineering or pavement management programs. A specific program for evaluating the benefits and cost of alternative treatments is presented in reference 1, Appendix C.
- d. Evaluation
- (1) Evaluation of completed projects as required in Highway Safety Program Standard No. 9 and FHPM 8-2-3, Highway Safety Improvement Program, should be well documented and should include a representative sample of completed projects. A sampling plan should be established, using accepted statistical methods, to evaluate projects with a range of such variables as classes of roadways, traffic volumes, types of countermeasures, pavements used, and other pertinent factors. On hazard elimination

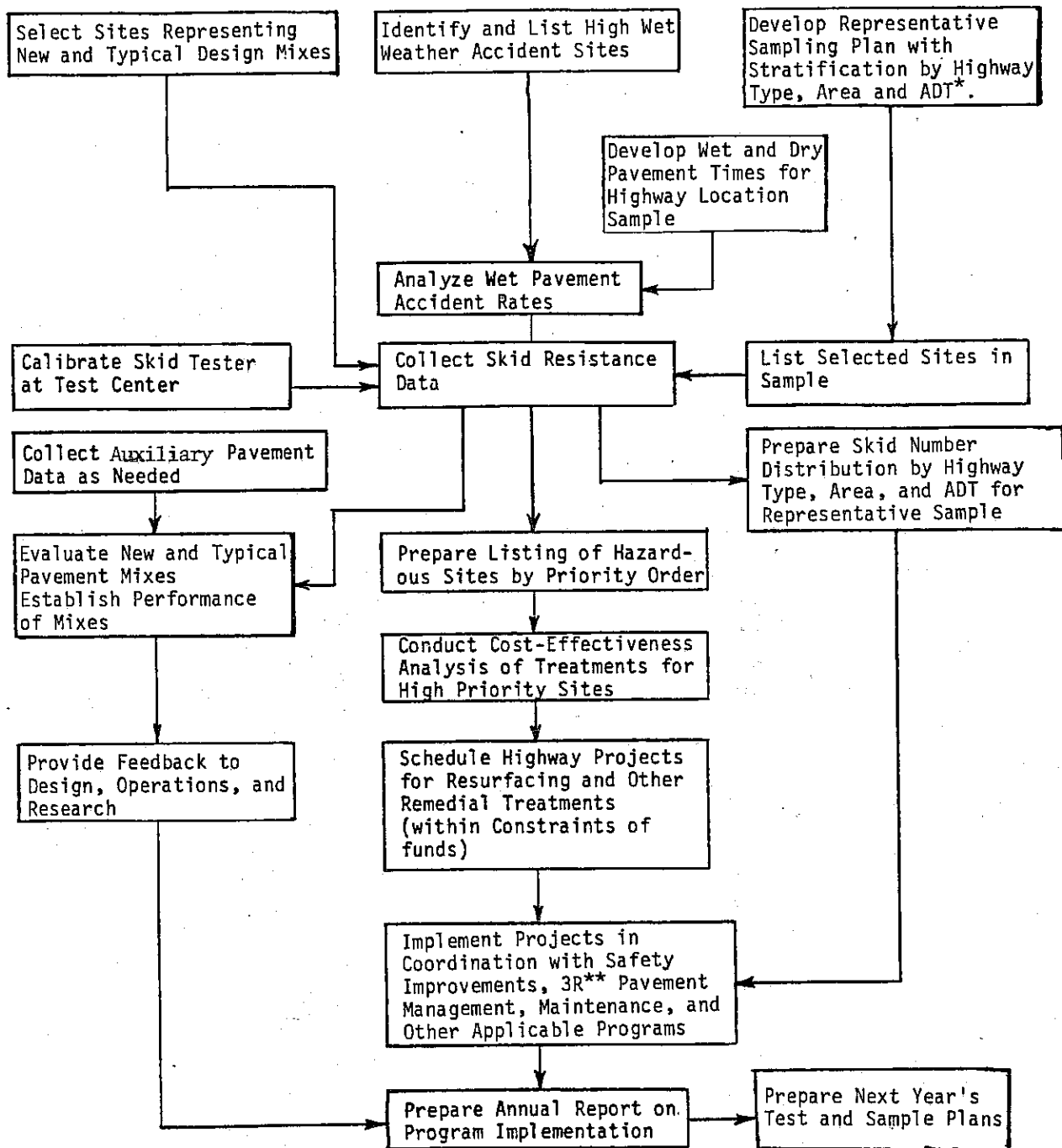
projects, these data should be correlated with accidents and traffic exposure and other pertinent factors in before/after analysis. See reference 2 in Appendix C.

- (2) The evaluation of completed safety projects should be a continuing process to ascertain the long-term performance of corrective actions such as skid resistant overlays. The evaluations should address at least:
 - (a) the overall effectiveness of the program in reducing accident rates at the corrected sites,
 - (b) the adequacy of the various materials, designs, or methods used, and
 - (c) recommendations for changes in the program, practices, or needed research and development.
- (3) As a secondary benefit, the evaluation process should provide input to an overall pavement management process.

6. PAVEMENT SKID RESISTANCE TESTING PROGRAM

- a. General Description of Program. The actual testing of pavement friction provides basic data for use in the three activities introduced in paragraph 3. Figure 1 graphically presents the interrelation between these activities. The upper portion of Figure 1 provides an overview of data to be collected to serve the safety, construction, and maintenance functions of highway organizations concerned with the skidding properties of pavement surfaces. The lower portion of Figure 1 indicates the various uses of the skid testing data, along with weather and accident data. Some of these data are evidence of the durability of particular surfaces, while other data provide a general overview of the skid resistance characteristics of the highway system.
 - (1) Skid resistance testing should be organized to support the following activities:
 - (a) Pavement evaluation studies in which measurements of the skid resistance of test sections are made to determine the skid characteristics of typical mix designs. Sufficient numbers of measurements should be

Figure 1
MODEL SKID ACCIDENT REDUCTION PLAN



* ADT: Average Daily Traffic

**3R: Resurfacing, Restoration and Rehabilitation

made to determine the level of pavement friction, wear rates, and speed gradient of the pavement under various traffic exposures. These test sections should include the new projects to be tested as described in paragraph 4b(4).

- (b) Evaluation of friction characteristics at locations which have a high incidence of wet weather accidents.
 - (c) System status for which measurements of the skid resistance of a representative sample of roads are made to develop the general levels of pavement friction on all roads in the highway agency's jurisdiction.
- (2) Accurate location of sites or road sections requires the use of a standardized reference system. Often each element of the State which collects highway data uses its own reference system. For example, police accident reports may locate accidents by distance to a landmark, pavement records may be kept by project number and geometric features may be identified by station. A unified reference system has many benefits, especially in pulling together technical data for identifying and analyzing locations with a high incidence of wet weather accidents.
 - (3) Pavement evaluation study sites and wet weather accident sites should be identified by the element within the highway agency responsible for those programs. The skid testing can then become a routine matter for the element charged with operation of the skid test equipment.
 - (4) A total skid inventory of all roads and streets in a highway system has proven to be impractical and is not necessary to carry out an effective skid accident reduction program. Roads and streets which are used primarily by vehicles traveling at low speeds are not highly susceptible to skid accidents and accordingly can be eliminated from routine sampling of highway sites. For urban areas, this means that most city arterials would be sampled but residential streets and roadways with low speed limits would not. Nearly all rural highway sections could be sampled, since such roads are liable to high-speed use.

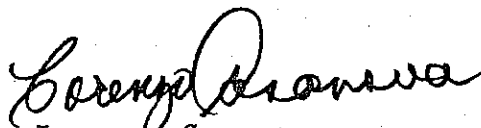
- (5) Another practical consideration in determining which roads should be sampled is traffic volume. In urban areas, most roads with high speeds have moderate to high traffic volumes whereas this is not the case for rural highways. Relatively few rural roads are used by more than 1,000 vehicles per day. On a cost-effectiveness basis, such roads can seldom justify resurfacing on the basis of safety considerations alone; therefore there is little benefit in routine sampling of low-volume rural roads.
- (6) Highway sections within the constraints of higher speeds and volumes need not be tested every year, since few roads vary substantially in skid resistance in any two or three-year period. Beyond this period, however, roads may lose significant skid resistance and may pose a serious danger to users. Using these criteria as part of a sampling plan will permit most if not all highway agencies to make maximum use of skid resistance data without increasing the amount of skid testing undertaken.
- (7) Skid resistance measurements should be made with a calibrated locked-wheel skid tester using the ASTM E 274 method and supplemental procedures described in Appendix B or an acceptable alternative method. Locations such as intersections and sharp curves which are not easily measured with the locked-wheel skid tester at the standard speed of 40 miles per hour should be tested at a lower speed. Such tests should be supplemented with texture measurements to permit extrapolation of available skid resistance to operating speeds. Alternative methods of measuring pavement friction properties may be used provided they correlate well with the locked-wheel skid tester.
- (8) In analyzing the skid numbers obtained, the time of year the measurements were taken has to be considered. Several States have published the results of their analyses and have developed methods for correcting skid number measurements taken during various periods and for different pavement surface types. See references 5 and 6 in Appendix C.

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December 23, 1980

- b. Specific Data From Sample Sites. In conjunction with skid resistance measurements, pavement wet time and accident records are desirable for each roadway section in the sample. The highway location system should be used for correlating data from different sources. An example of specific data which is desirable at each sample site is given in Appendix D.
- c. Sites With Low Skid Resistance. When sites with low skid resistance are identified during the testing of system status, these sites should be analyzed for corrective action. This can be done through a pavement management program, a high hazard elimination program, or other efforts. If the high hazard elimination program is used, the analysis should be in accordance with FHPM 8-2-3.



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Appendixes

EVALUATION OF WET PAVEMENT TIME AND ACCIDENT DATA

- A.1 The quantity of rainfall (inches) recorded by weather stations may be used to calculate the percentage of pavement wet time. Wet pavement time (WPT) may be estimated from total annual rainfall in inches (AR) as follows:*

$$WPT = 3.45 \ln (AR) - 5.07$$

Dry pavement time may be estimated by subtracting the amount of wet time and ice and snow periods from the total time in the period analyzed. Data from rainfall stations maintained by the National Oceanic and Atmospheric Administration's Weather Service may be used for wet and dry pavement time estimates for various areas within a State.

Isohyetal maps may be used to develop site wet pavement times. If ice and snow cover pavements for a significant portion of the time, a map for dry time should be prepared as well. Figure A-1 provides an example of a wet time map drawn from isohyetal charts.

A.2 Wet Safety Factor (WSF)

There are a number of ways to evaluate the relative safety of the subject location, one of which is the wet safety factor (WSF) approach.** For each wet weather accident location, a WSF may be developed. This factor is expressed as follows:

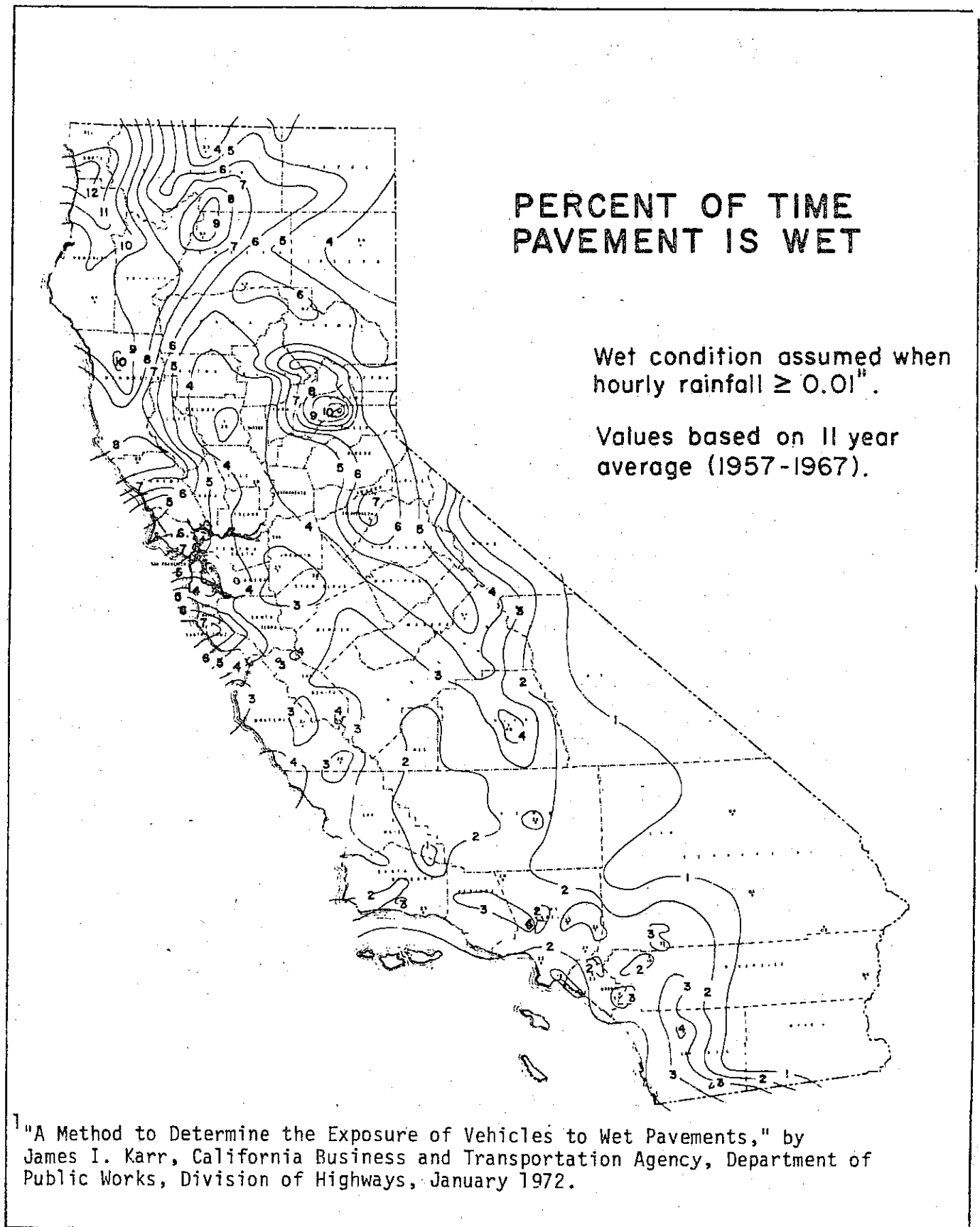
$$WSF = (DA)(PWT) / (WA)(PDT)$$

where: DA = number of dry weather accidents
WA = number of wet weather accidents
PDT = percent of dry pavement time
PWT = percent of wet pavement time

* This equation is based on a relationship developed by K.D. Hankins in "The Use of Rainfall Characteristics in Developing Methods for Reducing Wet Weather Accidents in Texas," Texas State Department of Highways and Public Transportation Study No. 135-4, July 1975.

** The WSF is a generalized form of an index referred to as the "skid trap ratio" and recommended for use in NCHRP Report 37, Tentative Skid-Resistance Requirements for Main Rural Highways," by H. W. Kummer and W. E. Meyer, Highway Research Board, Washington, D.C., 1967.

Figure A-1



This factor is the reciprocal of the risk of having a wet pavement accident relative to having a dry pavement accident. On a specific roadway section, each of these variables must be developed for the same time period; otherwise, traffic exposure must be taken into account. Criteria may be developed for further consideration of pavement sections. A WSF less than 0.67 suggests a wet weather problem. This criteria is based upon the conservative estimate of the overall likelihood of a wet weather accident being 1 1/2 as great as a dry pavement accident. This estimate assumes that wet weather accidents at the site or road section under consideration are attributable entirely to a skidding problem. A low WSF in most cases is due to poor skid resistance. However, traffic engineering evaluations may reveal deficiencies in sight distance, road markings, inadequate drainage, etc. Auxiliary information obtained during the test program should provide indications of the safety problems.

SKID MEASUREMENT SYSTEM DESCRIPTION AND OPERATING PROCEDURES

B.1 DESCRIPTIONS OF SKID MEASUREMENT SYSTEM

The requirements of American Society for Testing and Materials (ASTM) E 274 states "The method utilizes a measurement representing the steady state friction force on a locked test wheel as it is dragged over a wetted pavement surface under constant load and at constant speed while its major plane is parallel to its direction of motion and perpendicular to the pavement."

Although this specification may be met by a system involving only one wheel attached to a towing vehicle and although a few such systems are in use, the vast majority of skid measurement systems in use and expected to be in use in the near future consist of a towing vehicle and two-wheel trailer. On many systems either wheel may be locked during testing, but most commonly, the left is used.

The ASTM considers testing the left wheel track to be "normal." However, a differential in friction levels between the left and right wheel track may exist. When testing a site where a differential may exist, especially a high wet weather accident site, all lanes and wheel tracks should be tested. If a two-wheel trailer system is used, it is desirable to have the capability of testing with either wheel.

A skid measurement system must have a transducer associated with each test wheel which senses a force equal or directly related to the force developed between the sliding wheel and the pavement during test, electronic signal conditioning equipment to receive the transducer output signal and modify it as required, and suitable analog and/or digital readout equipment to record either the magnitude of the developed force or the calculated value of the resulting skid number (SN).

The system must include a facility for the transport of a supply of water--usually 200 to 500 gallons--and the necessary apparatus to deliver a specified amount of water--4.0 gallons per minute per wetted inch of pavement at 40 miles per hour within specified limits in front of the test wheel.

Finally, the system must include provision for measuring (and preferably for recording) the speed at which the test is conducted.

B.2 FIELD OPERATING PROCEDURES

B.2.1 Field Force Verification

It is generally impractical to perform force plate calibrations at frequent intervals while the measurement system is in the field. Facilities should, however, be available to permit the operator to ascertain that significant changes have not occurred in the force measurement subsystem since the most recent force plate calibration.

If the measurement system uses a torque transducer and is adaptable to mounting a torque arm, the verification can be accomplished within a reasonable time and effort. This device, consisting of an arm capable of being bolted to the test wheel in a horizontal position and of supporting known weights located at specified distances from the center of the test wheel, may be used to test the torque transducer to predetermined values of torque. Typically, the test wheel of the inventory system is raised off the ground, the torque arm is attached to the test wheel and held in a horizontal position, the brake of test wheel locked, and a series of known weights are suspended on the torque arm. This procedure will induce a series of known strains on the transducer, resulting in a series of output signals through the signal conditioning equipment. The magnitude of these signals should then be compared to the magnitude of signals produced through use of the same technique immediately after the most recent force plate calibration. Adjustment of signal conditioning equipment gain setting may be made to offset small force measurement subsystem variations which could occur.

Verification should be repeated periodically.

B.2.2 Test Tire and Wheel Preparation, Control of Tire Pressure

Tire Specification

Unless otherwise specified, all tests shall be performed with tires meeting the requirements of ASTM E 501, Standard Tire for Pavement Skid Resistance Tests, and all pertinent sections of that specification as well as ASTM E 274 should be observed in their use.

Tire Mounting and Break-in Procedure

The tire should be mounted on a Tire and Rim Association 6JJ rim. The rim should have been examined to determine that it has suffered no damage or misalignment in prior use. After mounting, and before break-in, the tire and wheel should be balanced. The tire should be subjected to a break-in of 200 miles use before being used for testing. This break-in may be accomplished by using the tire on the skid trailer wheel which is not used for testing. If the tire must be remounted before test use, it should be rebalanced after remounting.

Tire Warm-Up Procedure

The test tire should be inflated to $24 + 0.5$ pounds per square inch measured at ambient temperature. After tire pressure measurement and adjustment, the tire should be subjected to a 5-mile warm up, travelling at conventional highway speeds, before tests are performed. The 5-mile warm-up should be repeated on any occasion when the measurement system is parked for a period of 15 minutes or more.

Tire Wear and Replacement Procedure

The standard pavement test tire has a series of visual wear guide sipes (small circular holes) cast into each of the outer ribs of the tire. The test tire should be withdrawn from testing use when wear has progressed to a point at which the wear guide sipes are no longer visible. During routine testing, test tires should be examined at least twice daily (and more frequently as tire nears unacceptable wear level) to determine that wear has not progressed beyond acceptable limits.

Additionally, after any series of tests on pavements having very high skid numbers (in excess of SN=70) or in the event of a deliberate or inadvertent dry skid, the test tire should be examined for the development of a flat spot. If a significant flat spot or spots develop on a test tire, it should be withdrawn from test use due to the tendency of the test wheel to seek out and return to such a flat spot in subsequent lockups.

B.2.3 Watering Subsystem Procedures

Daily Procedures

Prior to the beginning of each day's activity, the crew should perform at least the following functions with respect to the water subsystem:

1. Determine that the water nozzle (nozzles) when in the testing position assumes the proper angle with respect to the pavement (ASTM E 274 requires an angle of 25 ± 5 degrees).
2. If the measurement system has provision for raising and lowering the nozzle between tests, determine that the mechanism is working properly and that the nozzle assumes a fully lowered position during the test sequence.
3. Determine that the nozzle, when in the test position, will discharge water directly in front of and centered on the test wheel.
4. Examine the nozzle outlet orifice to determine that it is free from damage or distortion.

The above inspections should be repeated during a day's testing in the event of operation on very rough highways (or in the event of any off-highway travel) which may have caused damage to the nozzle or adversely affected its orientation.

Water Trace Width Check

Periodically the crew should make a measurement of the water trace width as a gross measure of overall water subsystem performance. This may be accomplished by driving the measurement system over a pavement at a selected convenient speed (the same speed should be used on all occasions), initiating water flow without locking the test wheel brakes, and measuring the width of the resulting water trace on the pavement. The trace width measurement should be made as quickly as possible after passage of the inventory system (preferably within 30 seconds). This would require that one member of the crew drive and operate the measurement system while the other member is positioned off the side of the pavement at the location at which the measurement is to be made. Best results are achieved if this procedure is performed on a relatively smooth pavement surface (low macrotexture).

B.2.4 Instrumentation Calibration Verification

Provision should be made to allow for verification of the signal conditioning instrumentation calibration (to account for the effects of zero and gain drifts).

General Requirements for Calibration Signal

The minimum acceptable facility for verification of conditioning instrumentation is a calibration signal subsystem. The calibration signal should be provided from such a source and in such a manner that there is little likelihood of variation in the calibration signal itself. This assurance then permits the operator to make adjustments in the measurement subsystem gain to offset the frequent small deviations which occur due to changes in ambient temperature and other operating parameters.

Force Measurement Calibration Signal

The most straightforward technique for providing a force measurement calibration signal is to make provisions for switching a high quality shunting resistor of known value in parallel with one arm of the force transducer strain gauge bridge. This induces an imbalance in the bridge equivalent to the application of a known force to the transducer. The resultant signal is sufficient to verify, or provide means of adjustment for, all elements of the force measurement system forward of the transducer itself.

Frequency of Use

Instrumentation calibration verification through use of calibration signals should be accomplished at the beginning of each day's operation after equipment warm up, at intervals of no more than 2 hours when the system is in continuous use, and upon the renewal of operation throughout the day after any period during which the signal conditioning equipment has been turned off or the unit has been allowed to stand without use for 30 minutes or more.

B.2.5 Check List

A check list should be available to the crew and should be used prior to the beginning of daily operations and on any occasion during the day when testing is

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Appendix B

suspended for 30 minutes or more or when instrumentation has been turned off. The check list varies from system to system due to differences between the systems, but should provide for at least the following checks:

1. all power subsystems on and providing proper levels of power
2. all signal conditioning subsystems on for adequate time to reach stable operation (typically 10 to 30 minutes)
3. all recording systems on and functioning properly
4. instrument calibration (described above) performed
5. tire pressure checked and adjusted if necessary
6. test tire checked for wear
7. water nozzles checked for position and condition
8. water tank adequately filled
9. fuel supply adequate
10. safety chains and all other connections between trailer and towing vehicle properly connected, positioned, and protected if necessary
11. trailer jacks (if available) in retracted position
12. all auxiliary equipment (air-compressors, lights, etc.) functioning properly

B.3 USE OF STATIC AND DYNAMIC CALIBRATION PROCEDURES

B.3.1 Purpose of Field Test Center

At the present time the highest order of calibration and evaluation available for a State skid measurement system is that provided through the Field Test Center established under contract by the Federal Highway Administration (FHWA). Arrangements to receive the services of the Field Test Center may be initiated by a State through submittal of a request for such services to the local FHWA division office.

B.3.2 Criteria for When to Use the Field Test Center

Each measurement system should be submitted for calibration and evaluation at the Center as soon as possible after its introduction into service. It should be resubmitted for calibration and evaluation whenever:

1. significant repair or modification has been accomplished by the owning agency which might reasonably be expected to affect test results, or
2. whenever it has experienced sufficient use such that normal wear in the various subsystems might be expected to have affected their operation.

The second consideration suggests that each measurement system should be resubmitted at least every 2 years.

B.3.3 Calibration Services Provided by Field Test Center

The static and dynamic calibration services provided by the Field Test Center include the following:

1. Horizontal and Vertical Force Calibration. This provides for evaluation of the accuracy, linearity and hysteresis of the measurement system force transducers and signal conditioning equipment through use of an air bearing force plate maintained by the Center, and periodically calibrated by the National Bureau of Standards.
2. Flow Rate Evaluation and Adjustment if Required. This includes determination that the water delivery subsystem of the measurement system provides a quantity of water (dependent upon trace width) in front of the test tire which meets ASTM E 274 requirements at speeds between 20 and 60 miles per hour.
3. Static Evaluation of Water Distribution. This provides an evaluation of the uniformity with which the total water flow is distributed across the trace width and adjustment, if necessary, to assure that the water is in fact delivered uniformly and in line with the test tire.

4. Force Plate or Load Cells. The visitors force plate used for routine checks of the force measurement subsystem can be calibrated while at the Center.
5. Speed Calibration. The speed measurement (and recording if available) subsystem is evaluated, calibrated and, where necessary and possible, adjusted to produce accurate speed measurement values over the range of 20 to 60 miles per hour.
6. Tire Pressure Gauge Calibration. This provides assurance that tire pressures in the test wheels and in the speedmeasuring fifth wheel (if used) can be accurately measured and set.
7. Dynamic Correlation. Two such correlations are conducted: The first with the measurement system in the "as arrived" condition and the second after all of the foregoing evaluations have been conducted and indicated adjustments accomplished. The first correlation results in the development of mathematical relationships between the measurement system and the Area Reference Skid Measurement System that permit data collected by the measurement system, prior to its visit to the Center, to be adjusted to a common base provided by the use of the Area Reference System. The second correlation permits the development of similar relationships which may be used to relate the results of subsequent testing to the Area Reference System base. The data from the second correlation also provide an estimate of the system measurement variance.

B.4 MAINTAINING SYSTEM INTEGRITY BETWEEN FIELD TEST CENTER CALIBRATIONS

Two basic types of procedures are available for determining that significant changes have not occurred in the measurement system since its most recent evaluation and calibration at the Center. These involve techniques for evaluating important subsystem performance and techniques for evaluating performance of the total system.

B.4.1 Techniques to Evaluate Subsystem Performance

As a minimum, the owner of each measurement system should maintain and periodically make use of facilities for evaluating the force, water, and speed measurement subsystem of the inventory system.

Evaluation of Force Subsystem

The force subsystem should be evaluated through use of a force plate. An air-bearing force plate is recommended since its action is such as to essentially eliminate the effect of friction in the plate itself. If an air-bearing force plate is not available, any of several commercial mechanical force plates may be used. If a mechanical device is used, precautions should be taken to assure that all moving parts (particularly load application screws and spherical or roller bearings) are well lubricated and that the lubricant is periodically removed and replaced.

To conduct an evaluation, the test wheel of the measurement system should be centered on the force plate, the test wheel brake locked, and known frictional forces introduced to the tire-force plate interface through appropriate motion of the force plate. Frictional forces should be both increased and decreased in a stepwise manner to allow for detection of possible hysteresis effects. The indicated force readout values for the system should then be plotted against known force input values. The resulting plotted calibration line should be evaluated for nonlinearity and hysteresis characteristics. Also actual readout values for known force inputs should be compared with those readout values determined from tests conducted with the same equipment after the most recent Center evaluation.

Evaluation of Water Subsystem

The most effective evaluation of the water subsystem to discern variations in performance is that of flow. Flow rate may be evaluated by raising the rear wheels of the towing vehicle, running the vehicle at an indicated speed of 40 miles per hour (or any other desired speed), collecting the water pumped through the system and out the nozzle during a measured time period, and calculating the flow rate in gallons. This procedure should be repeated at two or more speeds to evaluate linearity of the water delivery subsystem with test speed.

The Pennsylvania State University has developed a water rate flow tank which is circular in cross section and of such size that it fits easily into a standard manhole. The tank has a threaded opening in the bottom for drainage and a stop-plug with a long handle which permits the plug to be removed and replaced from the top of the tank after it is hanging in the manhole. It also has a scale calibrated in gallons on the inside of the tank. This tank may be suspended in a standard manhole, the measurement system positioned so that the nozzle will discharge directly into the tank, the rear wheel of the towing vehicle raised, and total flow measured at any desired speed. The only additional equipment required is a stopwatch.

Evaluation of Speed Measurement Subsystem

The speed measurement subsystem should be evaluated by operating the measurement system at various test speeds over a measured mile course. If the basic speed measure is done through the use of the tow vehicle speedometer or through a tachometer-generator driven by the tow vehicle or by a fifth wheel, then the vehicle should be driven over the measured mile course at a selected speed and the time of transit measured with a stopwatch. The actual speed, calculated from the distance and the elapsed time, is then compared to the indicated speed.

If speed measurement is based upon a pulse generator driven by a fifth wheel, the accuracy of the speed measurement is directly dependent upon the accuracy of the fifth wheel for distance measurement. To evaluate this subsystem, the fifth wheel tire pressure is adjusted until the distance indicated agrees with the known distance traversed (the assumption being made here is that the electronic package which converts the pulses to velocity is functioning properly).

If tapeswitch event detectors, placed 200 feet apart, and an interval timer (+0.01 second resolution) are available to measure the time required by the inventory system to travel 200 feet, a very accurate speed measurement is obtained to check against the indicated value.

Time Between Subsystem Evaluations

The force, water and speed measurement subsystems of the measurement system should be checked by the methods described above at intervals no greater than 3 months.

B.4.2 Techniques to Evaluate Total System Performance

Use of Measurement System Sample Variance as Performance Measure

A portion of the information furnished, as a result of an evaluation at the Center, is the pooled sample standard deviation of the measurement system for repeated test at three test speeds on five special test surfaces. If the sample standard deviation at the desired speed is squared, the resulting value, SD_t^2 is an estimate of the skid measurement system variance. Subsequent to the Center evaluation, the crew should periodically select a pavement location having a skid number of approximately 30 to 40 and run 20 repeat tests at the desired speed over the same location. From the results of these latter tests, a new estimate, SD_E^2 , can be calculated. If the ratio SD_E^2/SD_t^2 does not exceed 2.0, the chances are 19 in 20 that the system standard deviation has not doubled over that established during its visit to the Center. (If the system has not been to a Center to obtain an estimate of SD_t^2 , its crew should select a pavement location having a skid number of approximately 30 to 40, run repeat tests at each desired speed over the same location, and calculate the sample standard deviation at each such speed.)

As an alternative, the above procedure could be performed making only 10 repeat tests on the selected pavement. In this case, the ratio of SD_E^2/SD_t^2 should not exceed 2.2. The chances are then four in five that the system standard deviation has not doubled over that previously established.

The above procedure should be performed at time intervals no greater than 3 months.

Short Term Checks of System Performance

The agency operating the measurement system should select several pavements located close to the site at which the system is normally garaged and perform repeated tests on the surfaces at quite frequent intervals, preferably weekly. Measured values of skid resistance on these surfaces will obviously change as the surfaces change from traffic wear, environmental, and/or seasonal variations. However, these changes

SPECIFIC DATA TO BE REPORTED FOR SAMPLE SITES

The following data should be collected in testing sample locations:

- D.1 Skid numbers (SN) should be taken for major classes of roads stratified by traffic volume and geographical location.
- D.2 Auxiliary data which should be included in order to establish distribution of skid numbers may include the following:
 - (a) Location of site or roadway section
 - (b) Responsible jurisdictional unit and route number or other designator
 - (c) Functional classification of road (e.g., two-lane, four-lane divided without full control of access, etc.)
 - (d) Surface type (e.g., bituminous, open-graded, concrete, fine finish, etc.)
 - (e) Average annual daily traffic (use traffic count data if available)
 - (f) Length of roadway section
 - (g) Lane where skid measurements are made
 - (h) Date of skid measurements
 - (i) Number of tests made in section
 - (j) Average SN
 - (k) Range of SN measurements
 - (l) Presence of atypical geometric or feature
 - (m) Evidence of skidding (e.g., skid marks, scarred posts, etc.)

REFERENCES

The following is a selected list of references which may be helpful in implementing the program described in this Technical Advisory. This list is not intended to be a bibliography of all documents available in this field:

- *1. Effectiveness of Alternative Skid Reduction Measures, Benefit Cost Model, Report No. FHWA-RD-79-12, Volume II, November 1978, Federal Highway Administration.
- *2. Accident Research Manual, FHWA-RD-80-016, February 1980, Federal Highway Administration.
- *3. Evaluation of Minor Improvements (Part 8), Grooved Pavement (Supplemental Report) CA-DOT-TR-2152-11-75-01, September 1975, R. N. Smith and L. E. Elliott, Office of Traffic, California Department of Transportation.
- 4. Evaluation of Minor Improvements (Part 9), Open Graded Asphalt Concrete Overlays, January 1972, James I. Karr, Office of Traffic, California Department of Transportation.
- 5. Variations in Skid Resistance Over Time, FHWA-VA-80-33, February 1980, S. N. Runkle, David C. Mahone, Virginia Highways and Transportation Research Council.
- 6. Seasonal Variations in the Skid Resistance Pavements in Kentucky, Research Report 532, November 1979, James L. Burchett, Roland L. Rizenbergs, Kentucky Department of Transportation.

* These studies are available through the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161

should occur in an orderly and predictable fashion and any abrupt change would be an indication of possible erratic performance of the measurement system. A continually updated record of the results of such tests should be maintained and examined after each updating for evidence of such erratic performance.

State of Illinois
Department of Transportation

SPECIAL PROVISION
FOR
OPEN-GRADED ASPHALT FRICTION COURSE

Effective January 2, 1980

Description. This work shall consist of the construction of an open-graded asphalt friction course on a prepared or existing bituminous concrete binder or surface course.

Materials. Materials shall meet the requirements of Article 406.02 except as follows:

Note 1: Coarse Aggregate. Any of the following coarse aggregates may be used to produce OGAFRC mixtures.

1. Description

(a) Crushed Steel Slag. Crushed steel slag shall be the graded product resulting from the processing of steel slag from an open-hearth or basic-oxygen furnace.

(b) Crushed Slag.

(c) Crushed Trap Rock.

2. Quality. The aggregate shall have Class B quality or better meeting the requirements of Article 704.01(b), except the Los Angeles Abrasion Test, AASHTO T 96, does not apply to crushed slag aggregate.

3. Gradation. The coarse aggregates shall be uniformly graded from coarse to fine and when tested by means of laboratory sieves (square openings) shall conform to the following gradation. A maximum of 20 percent fine aggregate may be blended with the coarse aggregate to obtain the required gradation.

Passing 1/2" sieve -----	100%
Passing 3/8" sieve -----	90-100%
Passing No. 4 sieve -----	30- 50%
Passing No. 8 sieve -----	10- 18%
Passing No. 200 sieve -----	2- 5%

Note 2: Bituminous Materials. Asphalt Cement grade 70-85 shall be used. The Contractor shall use an approved heat-stable anti-stripping additive. The anti-stripping additive shall meet the approval of the Engineer based on the results of laboratory tests conducted by the Bureau of Materials and Physical Research. The additive shall be added to the asphalt tank at the recommended dosage (0.5 to 1.0 percent by weight of asphalt cement) and shall be thoroughly mixed by circulation of the asphalt for at least four (4) hours prior to being incorporated into the mix. The exact amount of additive shall be determined by the Engineer based on laboratory tests.

Equipment. Equipment shall meet the requirements of Article 406.03, except that the hot-mix plant shall be a batch type plant. The use of hot-mix surge bins will not be permitted.

CONSTRUCTION REQUIREMENTS

General Conditions. Article 406.04 shall apply except the mixture shall be placed only when the daily high air temperature is at least 60° F. two days prior to placement and there is a forecast of high temperature of at least 60° F. during and for two days after construction. Official National Weather Service data for the construction area shall be used.

Keeping Road Open to Traffic. Article 406.05 shall apply.

Preparation, Priming and Leveling of Brick, Concrete or Bituminous Bases. Article 406.06 shall apply.

Preparation of Asphalt Cement. Article 406.08 shall apply.

Preparation of Mineral Aggregates. Article 406.09 shall apply except the aggregates shall be heated in such a manner as to assure that the mixing temperature is uniformly maintained. The aggregates shall be dried to less than 0.5 percent residual moisture by weight, as determined by hot bin samples. This may require the aggregate to be processed twice through the drier. The aggregate(s) shall be screened into at least two (2) sizes before mixing.

Mixing Formula. At least two weeks prior to the placement of any of these mixtures, the contractor shall furnish to the Engineer, samples of the aggregates he proposes to use. The Engineer shall perform mix design tests to determine the exact proportion for the mix which will be between the following composition limits by weight:

Aggregate-----	85 to 95%
Asphalt Cement-----	5 to 15%*

*Note: The range of asphalt content is based on the varying physical properties of the coarse aggregate that can be used for the manufacture of OGAF C. Upon request, the Engineer will provide the Contractor with an approximate asphalt content, $\pm 1\%$ for any given coarse aggregate. The amount of anti-stripping agent will not be included in this percentage.

Preparation of Bituminous Mixtures. Article 406.12 shall apply except, the mixing temperature shall not exceed 260° F.

Transportation of Mixtures. These mixtures shall be transported in covered and insulated trucks conforming to Article 406.13. The cover shall be rolled back before the load is dumped into the finishing machine. Covering may be waived on short hauls or in hot weather.

Placing of Bituminous Mixtures. Article 406.14 shall apply except for the following:

1. The mixture shall be at a temperature of 230° F. \pm 20° F. at the time of placement.
2. The mix shall be placed within one hour from the time of completion of mixing.

3. No straightedging will be required.
4. The paver speed shall be limited to not more than 35 feet per minute.
5. The mix shall be placed at a nominal thickness of 5/8" compacted.

Note: Approximate unit weights for Open-Graded Asphalt Friction Course mixtures.

(a) Crushed Steel Slag - 75 lbs/sq. yd. at 5/8" thick.

(b) Crushed Slag - 56 lbs/sq. yd. at 5/8" thick.

(c) Crushed Trap Rock - 66 lbs/sq. yd. at 5/8" thick.

Compaction of Mixtures. Immediately after placement of the mixtures, the pavement shall be compacted by two (2) tandem rollers conforming to Article 406.03, Note 3. No more than a total of three (3) coverages by the rollers will be required. When approved by the Engineer, vibratory rollers may be used in the static mode. More than three (3) coverages may be required when using vibratory rollers due to lower unit weights. The Engineer may eliminate one roller on small jobs. The amount of rolling shall be confined to only that necessary for consolidating the bituminous mixture and bonding it to the underlying surface. Excessive rolling shall be avoided.

Protection of Pavement. Article 406.21 shall apply. No traffic shall be allowed on any portion of the completed pavement until after the final rolling and the mixture has cooled adequately to prevent pick-up.

Method of Measurement. OPEN-GRADED ASPHALT FRICTION SURFACE COURSE shall be measured in place and the area computed in square yards. The width used shall be that which is shown on the plans.

Basis of Payment. This work will be paid for at the contract unit price per square yard for OPEN-GRADED ASPHALT FRICTION SURFACE COURSE measured as specified herein. If not provided as a payment item, the cost of preparation of the base shall be included in the cost of the bituminous surface.

State of Illinois
DEPARTMENT OF TRANSPORTATION

APPENDIX C

Bureau of Local Roads and Streets

SPECIFICATION
FOR
PLASTICIZED BITUMEN HOT MIX SEAL

DESCRIPTION. This item shall consist of the furnishing and spreading of a Plasticized Bitumen Hot Mix Seal mixture to a compacted thickness of three-fourths (0.75) inch or less all in accordance with the requirements of these specifications, and to the lines, grade, thickness and cross sections shown on the plans or established by the Engineer.

STANDARD SPECIFICATIONS. All references to Sections or Articles in this specification shall be construed to mean a specific Section or Article of the Standard Specifications for Road and Bridge Construction, adopted by the Department of Transportation.

MATERIALS. Material shall meet the requirements of the following articles of Section 700 - Materials.

<u>Item</u>	<u>Article</u>
(a) Aggregate (Note 1)	703.01, 703.03 (a, b & d)
(b) Bituminous Materials (Note 2)	Section 713

Note 1. The aggregate shall be a blend of stone screenings and sand combined to meet the following gradations:

Passing No. 4 Sieve	100%
Passing No. 10 Sieve	82 + 13
Passing No. 20 Sieve	60 + 20
Passing No. 40 Sieve	43 + 18
Passing No. 80 Sieve	17 + 8
Passing No. 200 Sieve	8 + 2

The method of blending shall be by the use of aggregate feeders of the apron, drum, reciprocating, or other type approved by the Engineer, which shall provide for proportional and total feeding of the aggregates. A minimum of 30 percent on the screening of each aggregate will be required. The components of a blend need not be of the same kind of material. The source of material and blending proportions shall not be changed during the progress of the work without written permission from the Engineer.

Stone Screenings. Stone screenings shall be of Class B Quality or better and produced by the secondary or tertiary crushing operation.

Sand. Sand shall be of Class B Quality or better. No more than 2.5% minus No. 200 material will be allowed in the sand.

Note 2. The contractor shall use one of the following grades of asphalt cement: AC 150-200; AC 120-150; AC 100-120; AC 85-100.

Asphalt Additive. An additive known as TAPISABLE, RECOPAVE, or approved equivalent, shall be added to the asphalt in the proportions recommended by the manufacturer.

Equipment. The following required items of equipment shall conform to Section 800-Equipment.

<u>Item</u>	<u>Article</u>
(a) Three-wheel Roller	801.01(f)
(b) Tandem Roller	801.01(e)
(c) Hot Mix Plant (Note 1)	802.01
(d) Spreading and Finishing Machine	802.03
(e) Motor grader with Special Spreading Blade	

Note 1. The bituminous hot mix plant shall be equipped with a circulating hot asphalt system to thoroughly mix the additive and asphalt prior to induction into the mix.

The bituminous mixture as specified herein shall be prepared in any type of Mixing plant.

CONSTRUCTION METHODS

General Conditions. The seal mixture shall be laid preferably on a base which is dry, and only when weather conditions are suitable. No mixture shall be laid when the temperature of the air in the shade is below 40°F.

Rolling shall be done with three-wheel or tandem rollers. The rollers shall weigh 8 to 12 tons and shall have a unit compression of not less than 200 nor more than 400 pounds per inch of roller width. Rollers shall be propelled at the rate of not more than 175 feet per minute.

All surfaces shall be cleaned of dirt, debris, and loose material prior to placing the mixture.

Keeping Road Open to Traffic. The road shall be kept open to traffic on the existing pavement or on the new work. During the actual cleaning of the pavement and the placing of the overlay, one-way traffic shall be permitted. At all other times, two-way traffic shall be allowed to use the road.

All barricades, warning signs, flags, and torches or lights shall conform to Article 107.14.

The Contractor shall keep all equipment, materials, and vehicles off the pavement and shoulder on the side of the pavement that is open to traffic.

Preparation of Base. When an existing bituminous concrete pavement is to be overlaid, all excess crack filler shall be removed. All bitumen shall be removed from cracks more than 1½ inches wide. The Contractor shall perform this work in the most economical manner practicable and as directed by the Engineer. All waste material placed on the shoulders during the pavement

cleaning operations shall be removed at the close of each day's work and shall be disposed of outside the limits of the right of way at locations acceptable to the Engineer. This work will be paid for in accordance with Article 109.04.

Prior to placing the seal mixture, all open cracks having a width of $\frac{1}{2}$ inch or more, cracks that have been cleaned, and depressions of one inch or more in the existing pavement or base, shall be completely filled with a bituminous mixture meeting the approval of the Engineer.

The mixture shall be tamped in place with hand tools. This work shall be completed at least 24 hours prior to placing the overlay.

Preparation of Aggregate and Bituminous Materials. The aggregate and bituminous materials shall be heated to the following temperatures.

Asphalt Cement Mixture

Aggregate	250°F. to 275°F.
Asphalt	250°F. to 275°F.

Preparation of the Mixture. Prior to the production of the mixture, the contractor shall furnish to the Engineer a mix formula with substantiating test data listing the source(s) of material, percent and gradation(s) of the aggregate(s), asphalt to additive ratio and the percent of the blend used in the mix, and the combined mix gradation, based on the recommendations of the additive manufacturer. Once the mix formula is approved by the Engineer, the mix will be prepared as directed by the following specifications. The heated aggregate and the asphalt for the seal mixture shall be measured separately and accurately. The bituminous mixture shall be made in the pug mill mixer. The time required to add the asphalt shall be not more than 15 seconds. The total time required to add the asphalt in a batch plant and complete the wet mixing period shall be not less than 30 seconds, or longer if necessary to produce a homogeneous mixture in which all particles of aggregate are coated uniformly.

The ingredients shall be heated and combined in such manner as to produce a mixture which when discharged from the mixer should not, in general, vary more than 20°F. from the temperature set by the Engineer; in all cases, the temperature shall not exceed 275°F.

The ingredients of the mixture shall be combined in such proportions as to produce a mixture meeting the following composition limits by weight:

<u>Ingredient</u>	<u>Per Cent by Weight</u>
Aggregate	90.0 to 94.0
Residual Bitumen	6.0 to 10.0

Transportation of Mixtures. Trucks used in transporting the bituminous mixture shall have capacities of not less than 4 tons. Trucks shall have tight dump bodies which have been previously cleaned of all foreign material and sprayed with distillate oil. The body of the truck shall be in a completely raised position when sprayed with the distillate oil, and it shall remain in this position until all excess oil has drained from the truck body. Unless

artificial light satisfactory to the Engineer is provided, no bituminous mixture which cannot be placed and compacted during daylight shall be delivered at the work.

Placing of Bituminous Mixtures. The bituminous mixture shall be placed true to crown and grade with either a spreading and finishing machine or a motor grader equipped with a special spreading blade with wings. The bituminous mixture may be spread and finished by approved hand methods only where machine methods are impractical, as in the case of special areas which because of irregularity, inaccessibility, or unavoidable obstacles do not lend themselves to mechanical placing.

Placing of the bituminous mixture shall be as continuous as possible. The base or existing surface shall be kept clean, and any foreign material shall be removed to the satisfaction of the Engineer before the seal is placed.

The spreading and finishing machine or a motor grader equipped with a special spreading blade with wings, shall spread the bituminous mixture without tearing the surface and shall strike a finish that is smooth, true to cross section, uniform in density and texture, free from hollows, transverse corrugations, and other irregularities. When the machine causes surface irregularities such as hollows or transverse corrugations, the machine shall be repaired or adjusted not later than the end of the day's work and it shall be in good working condition before work is resumed. The machine shall be operated at a speed that will insure as nearly as possible, continuous operation. The operating speed should not exceed 35 feet per minute for the finisher; a higher speed may be used if written approval is given by the Engineer but in no case shall the speed of the finisher exceed 60 feet per minute. If a motor grader is used for spreading the mixture, the speed shall be set by the Engineer but shall not exceed 200 feet per minute. If, in the opinion of the Engineer, the production of the plant exceeds the amount that can be laid satisfactorily with one finishing machine, the production shall be decreased or 2 machines shall be used.

The outside edges of the seal shall be sloped and pressed in place by means of a self-adjusting, constant pressure edge plate held in proper position on the spreading or finishing machine, except where the edges are supported by a curb, gutter or similar structure. A string line shall be used as a guide for the finishing machine in order to maintain a uniform edge alignment; if any other method is proposed it shall meet the approval of the Engineer before being used. No material shall extend beyond the limits of the base or existing surface.

Irregularities in alignment along the outside edges and along the longitudinal joint shall be corrected by adding or removing bituminous mixture before the edges are rolled. Excess bituminous mixture deposited outside the limits of the lane being laid shall be removed immediately and disposed of as directed by the Engineer.

Compaction of Mixture. Immediately after the mixture is placed it shall be compacted thoroughly and uniformly with a three-wheel roller or a tandem roller. Where initial rolling causes undue displacement, hair-cracking, or checking in the seal, the time of rolling shall be adjusted by the Engineer to correct these conditions.

One three-wheel roller and one tandem roller or two tandem rollers will be required on each project where the hourly production of the plant is 75 tons or less. One three-wheel roller and two tandem rollers, or three tandem rollers, will be required on each project where the hourly production of the plant is more than 75 tons.

Rollers shall be operated by competent and experienced rollermen and shall be kept in operation as continuously as possible so that all parts of the pavement will receive substantially equal compaction at the time desired. During each 8-hour day of laying bituminous mixtures, each roller shall be engaged in actual rolling for not less than $6\frac{1}{2}$ hours, and not more than $1\frac{1}{2}$ hours shall be allowed for refueling, watering, and similar work. Delays in rolling freshly placed bituminous mixtures will not be permitted.

Rolling of the first lane of seal to be placed shall start longitudinally at the edge having the lower elevation and progress to the other edge, overlapping uniformly on successive trips by at least $\frac{1}{2}$ the width of the rear wheels. Where laying the bituminous mixture adjacent to a previously placed lane, the first pass of the roller shall be along the longitudinal joint in such a manner that not more than $\frac{1}{3}$ the width of the rear wheel is on the freshly placed mixture; after which the rolling shall proceed from the outside edge toward the longitudinal joint, overlapping uniformly on successive trips by at least $\frac{1}{2}$ of the width of the rear wheels. Succeeding trips of the roller shall be terminated at least 3 feet from the preceding stop. Each stop shall be regulated to prevent trapping of water on the rolled surface.

The speed of the roller at all times shall be slow enough to avoid displacement of the bituminous mixture; if displacement occurs, it shall be corrected at once by raking and applying fresh bituminous mixture where required. To prevent adhesion of the bituminous mixture to the roller, the wheels shall be kept properly moistened but an excess of water will not be permitted.

Immediately after the initial rolling of the seal coat, the Contractor shall test the surface for smoothness with a 10-foot straightedge to locate high and low spots so that they may be altered while the mixture is still workable. Rolling of the seal shall be continued until all roller marks are eliminated and the bituminous mixture is satisfactorily compacted.

When required by the Engineer, the seal shall be rolled diagonally in two directions with a tandem roller, the second rolling crossing the lines of the first, and if the width of the pavement permits, it shall also be rolled at right angles to the center line.

In all places inaccessible to the rollers, such as locations adjacent to curbs, gutters, headers, manholes, and similar structures, the required compaction shall be secured with hot tampers.

Any bituminous mixture that becomes loose, broken, mixed with foreign materials, or which is defective in finish or density, or which does not comply in all other respects with the requirements of the specifications, shall be removed, replaced with suitable material, and finished in accordance with these specifications.

Protection of Pavement. The Contractor shall protect all sections of newly constructed seal from traffic until they have hardened to the satisfaction of the Engineer.

COMPENSATION

Method of Measurement. Plasticized Bitumen Hot Mix Seal shall be measured by weight in tons. The mixture may be measured either by weighing the mixture on platform scales approved by the Engineer or on the basis of plant weights. If measured on the basis of plant weights, an occasional check shall be made by weighing full truck loads of the bituminous mixture on an approved platform scale at the plant or on an approved commercial scale. When the method of measurement is by truck weight, the weight of each load shall be determined by weighing the truck each time before and after loading. If, during the course of construction, it becomes apparent that the weighman on the mixer platform or the weighman at the platform scale is not exercising proper care in weighing the bituminous mixture, he shall be removed at the direction of the Engineer and replaced by a competent and qualified workman. Quantities of materials wasted or disposed of in a manner not called for in the contract will be deducted from the final total measured quantities. The Contractor shall furnish approved duplicate load tickets upon which is recorded the net weight of the bituminous mixture in each truck. The tickets shall have sufficient space for signatures, identification of the bituminous mixture, date of delivery, and any other data which the Engineer may require. The Contractor shall submit one load ticket to the Engineer at the plant after the truck is loaded and the other load ticket to the Engineer at the work when the truck arrives.

Payment will not be made for seal mixture in excess of 105 per cent of the amounts specified by the Engineer.

Basis of Payment. This work will be paid for at the contract unit price per ton for PLASTICIZED BITUMEN HOT MIX SEAL measured as specified herein.

